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FEBRUARY, 1955. **Vol. L, No. 2**

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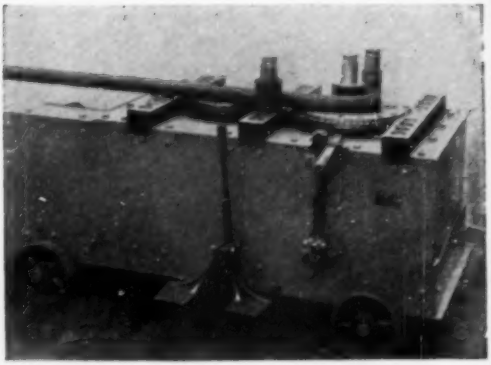

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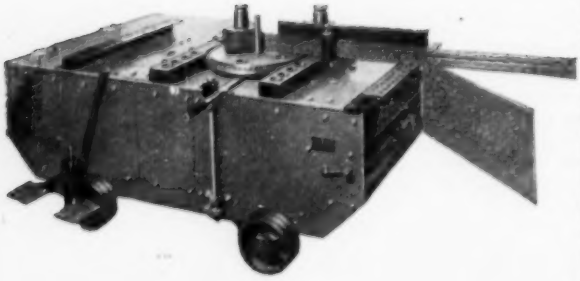

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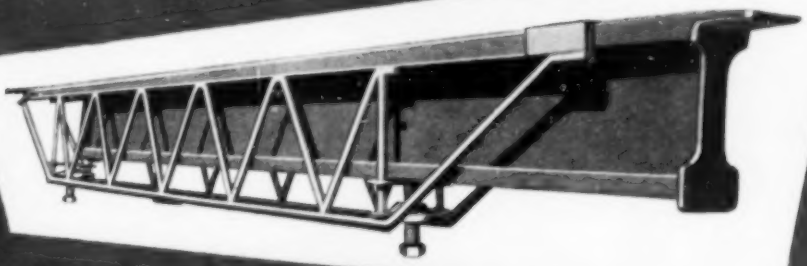

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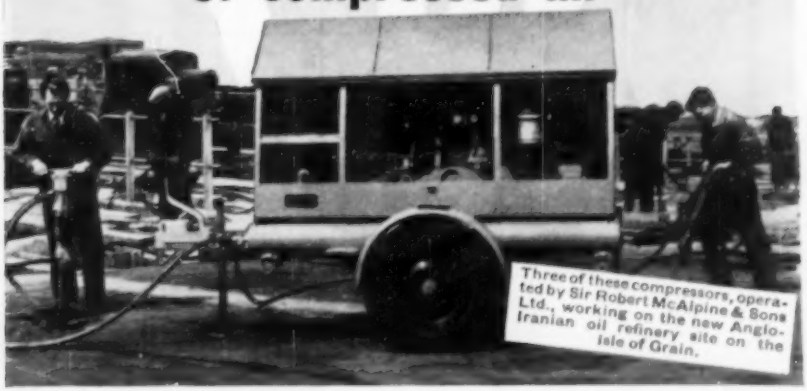

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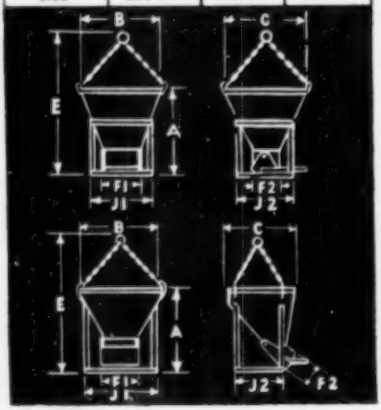

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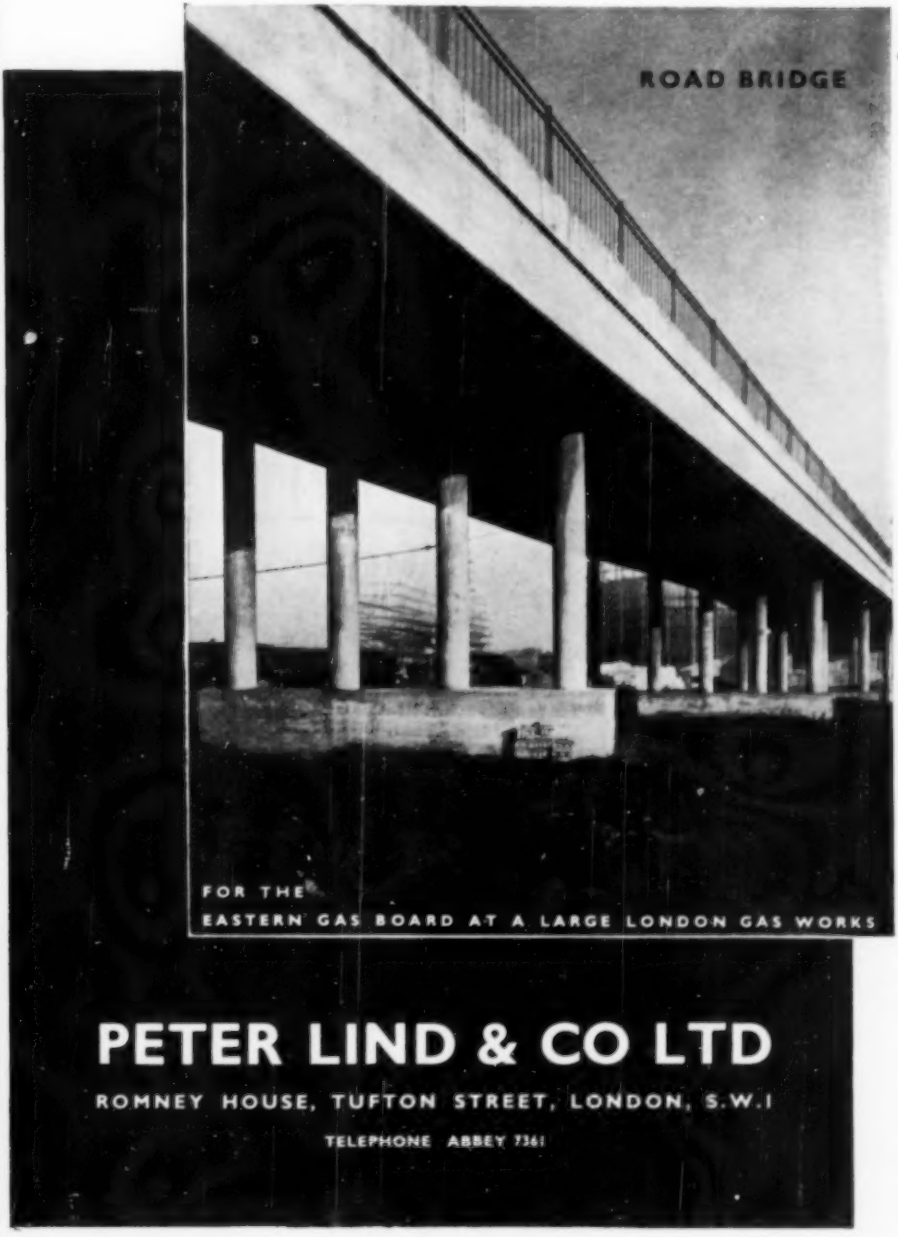

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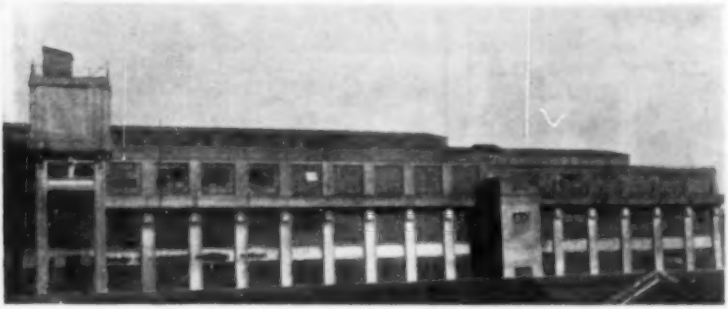

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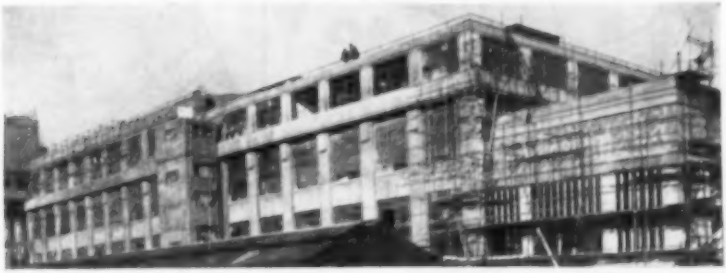

The reinforced concrete structure of Commercial Solvents (Great Britain), Ltd., at Bromborough, Cheshire (see top illustration), was built about 30 years ago, and had deteriorated through rusting of steel reinforcement with insufficient cover. The main renovation of this structure (see bottom illustration) included cutting-out defective concrete, removing rust-scale from exposed reinforcement, providing a new concrete surface of about 85,000 sq. ft., and fixing 1200 ft. of new coping. The exterior concrete surface treatment was carried out with the "Aerocem" cement spraying process.

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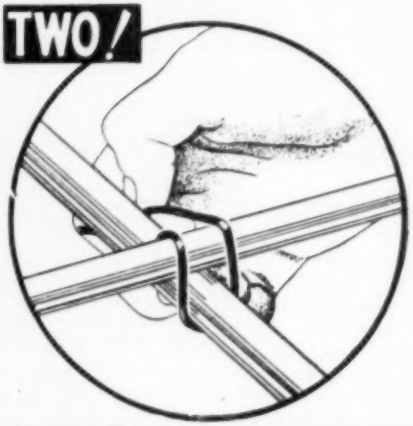

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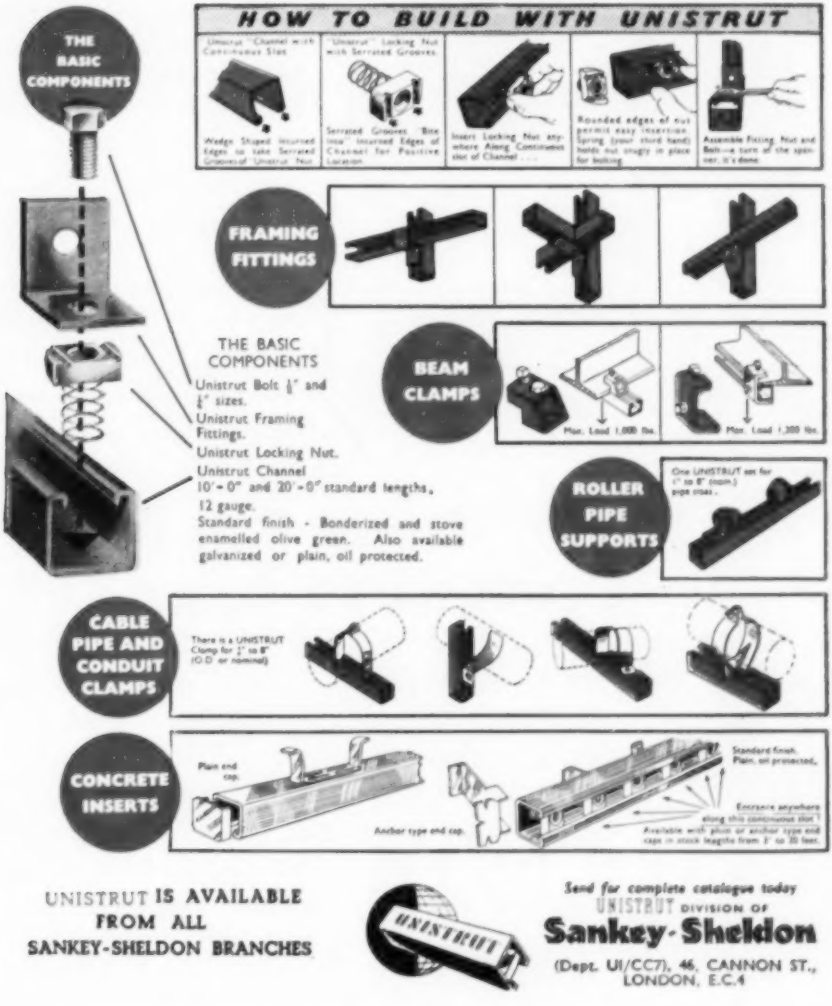
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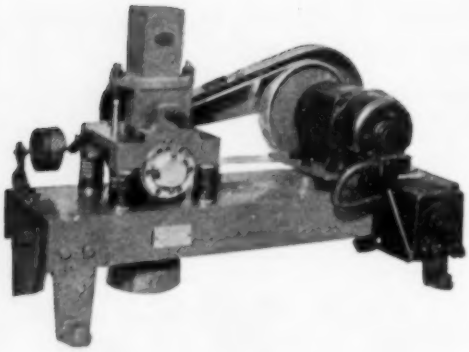

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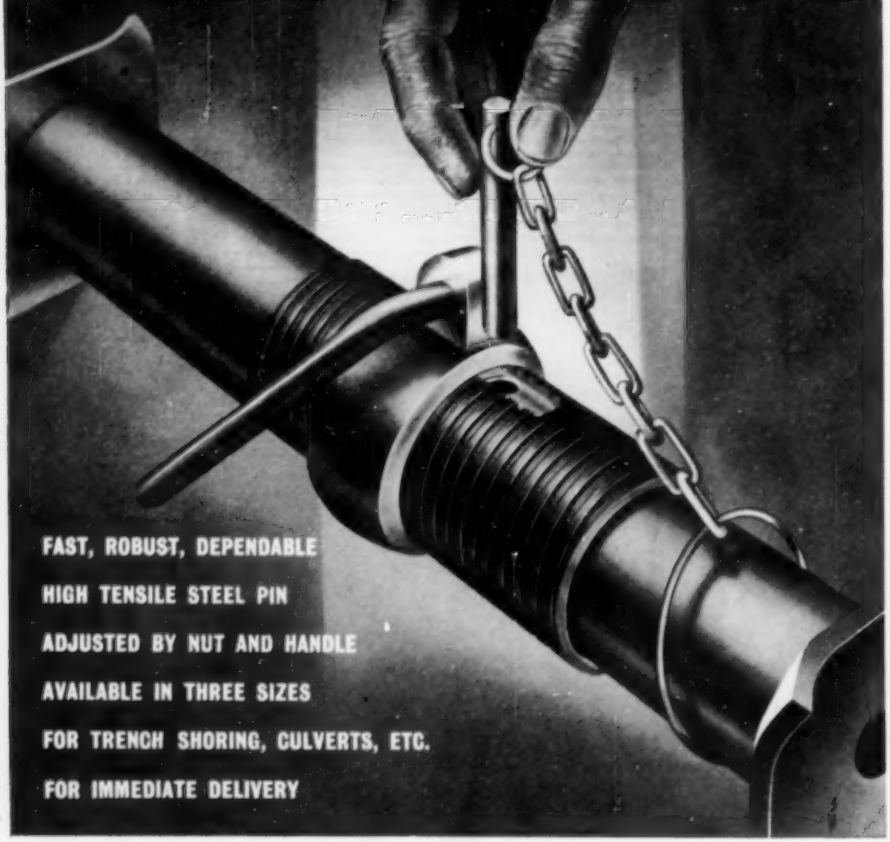
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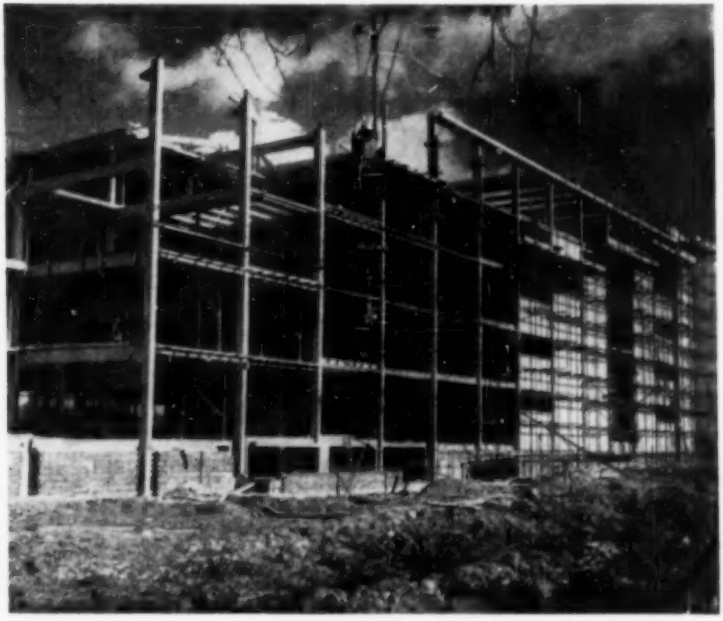

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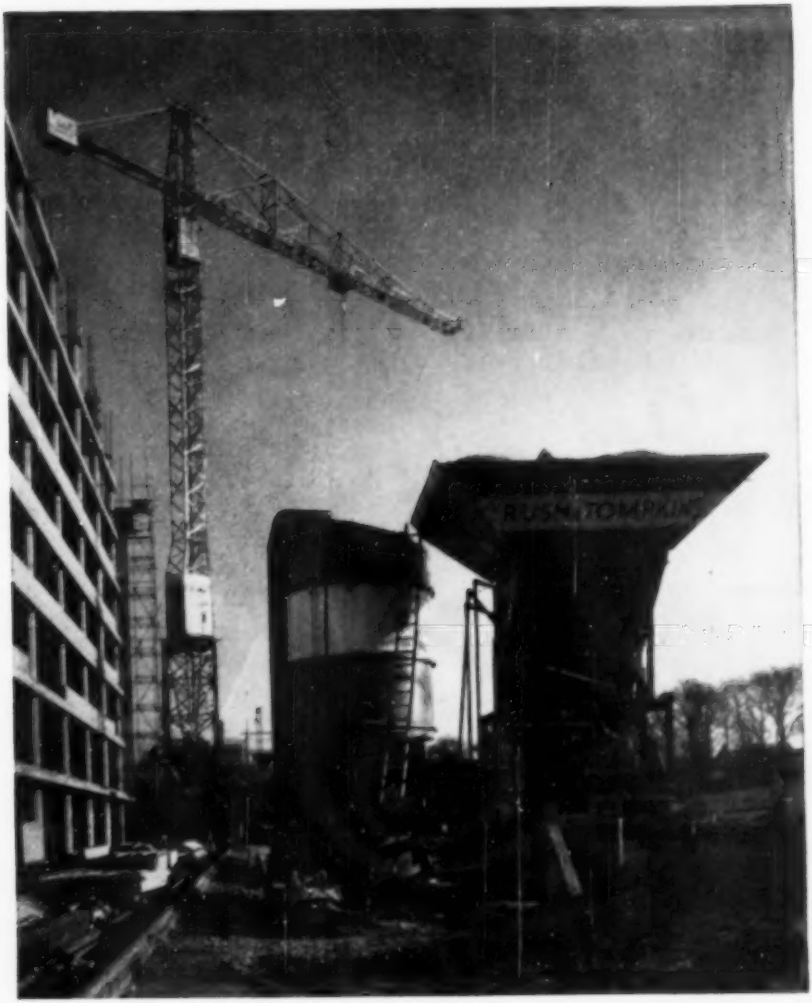

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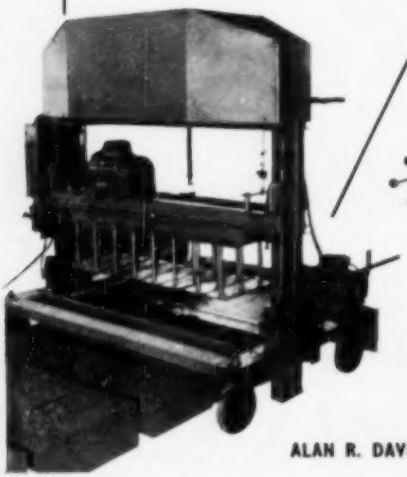

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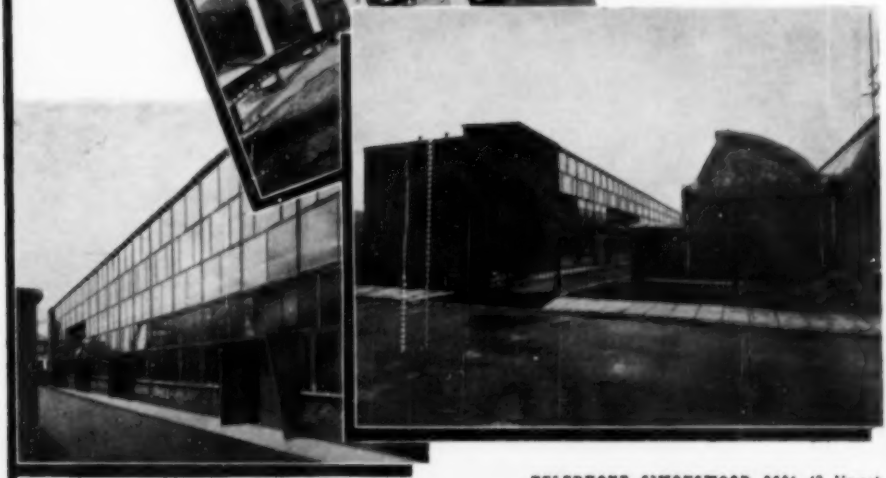

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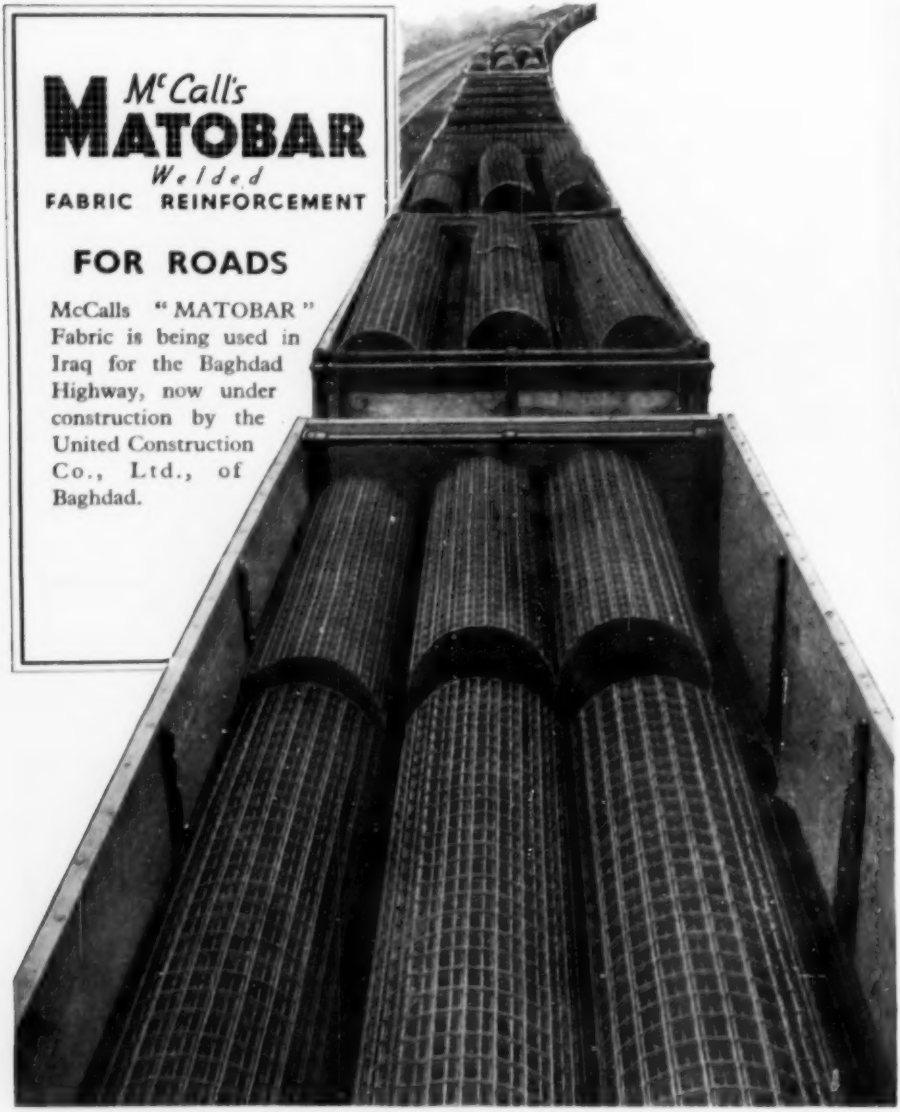

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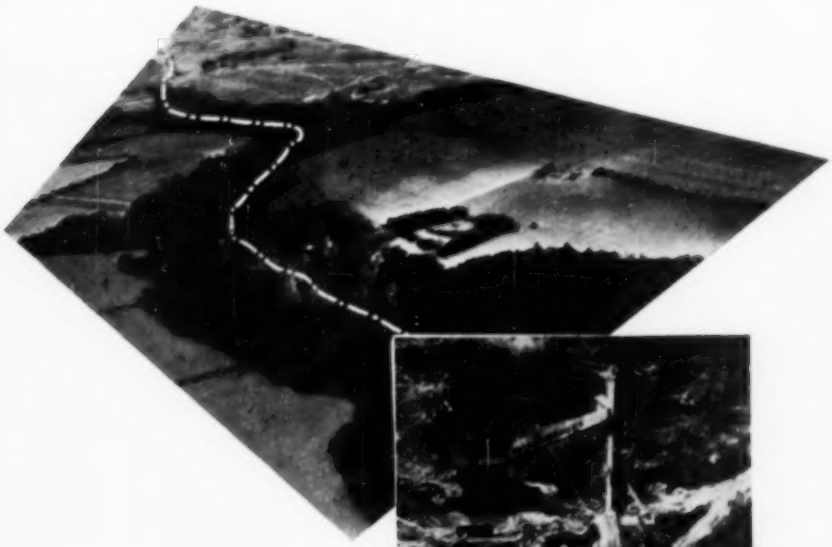

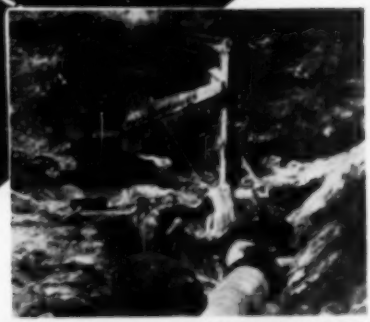

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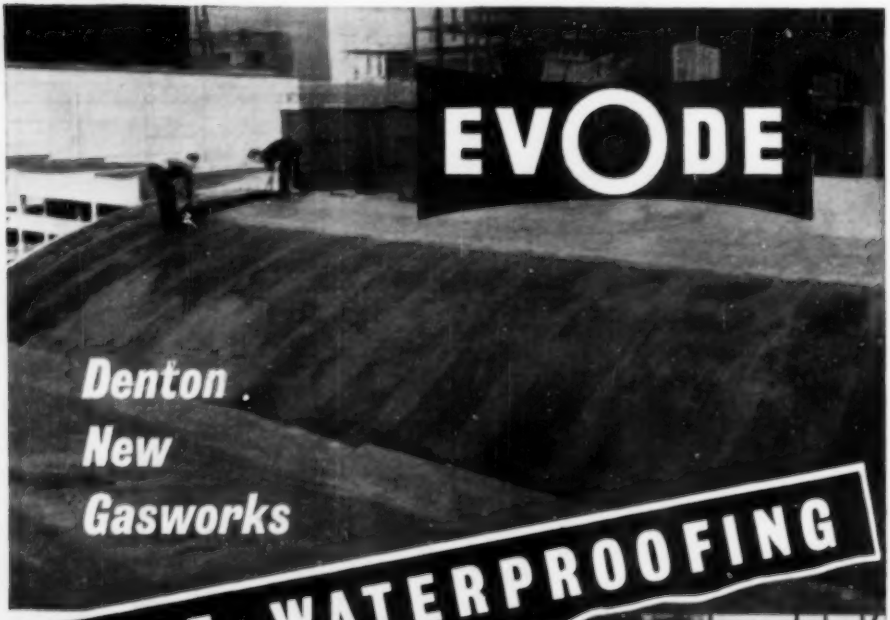

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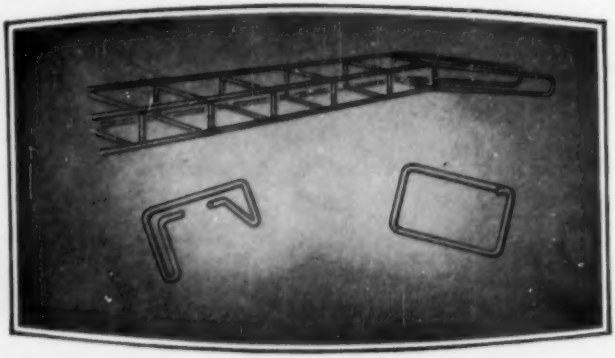

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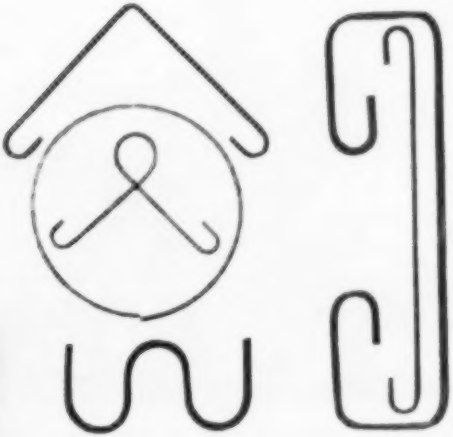

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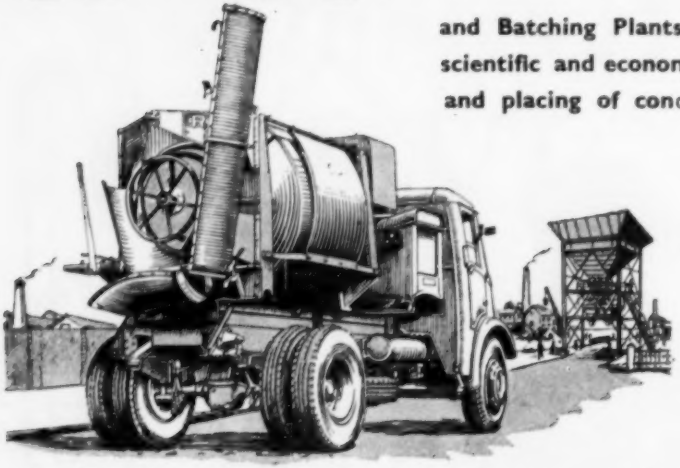

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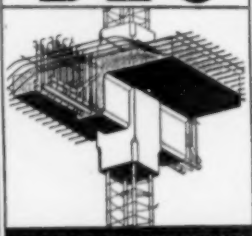

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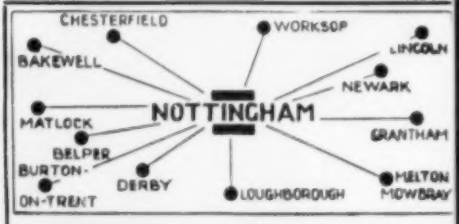

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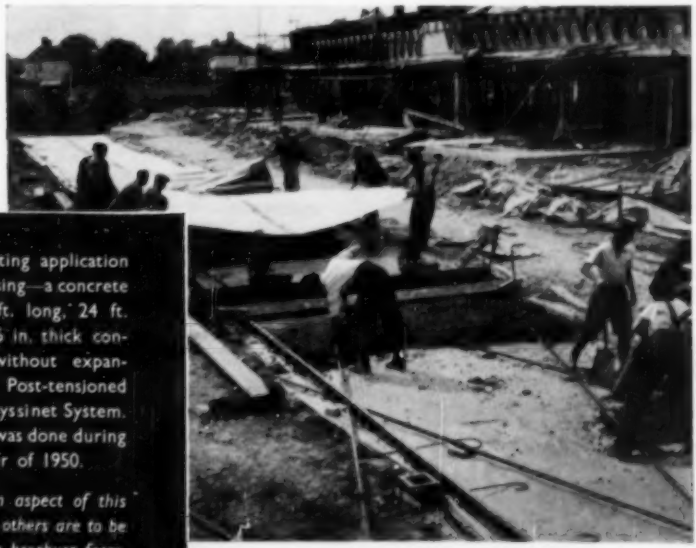

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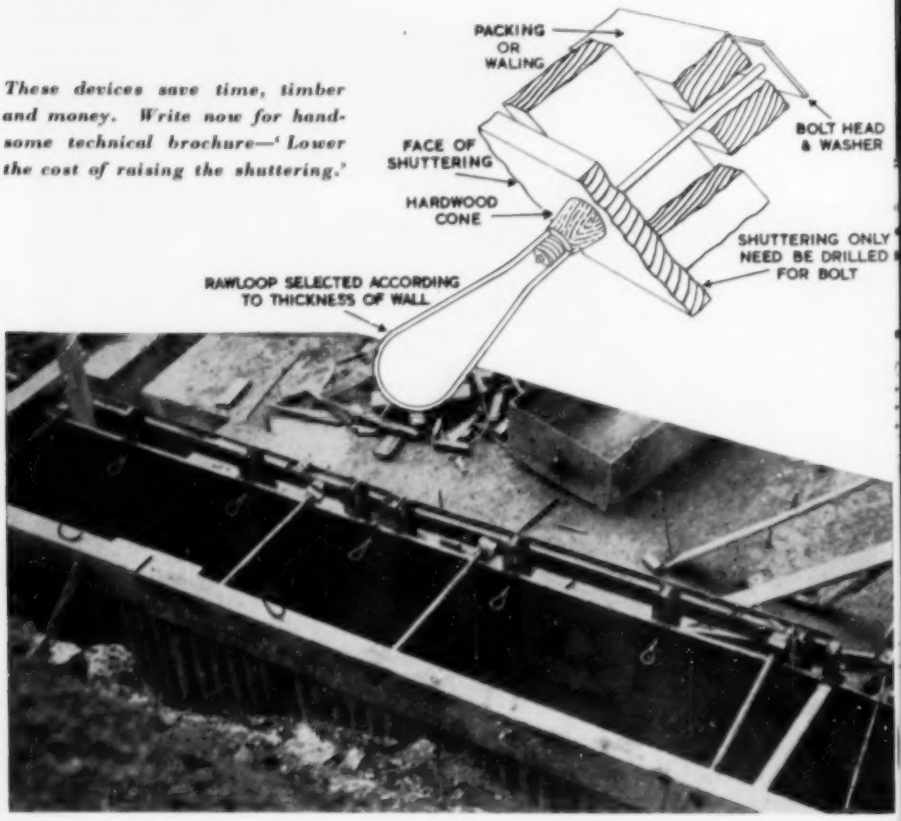
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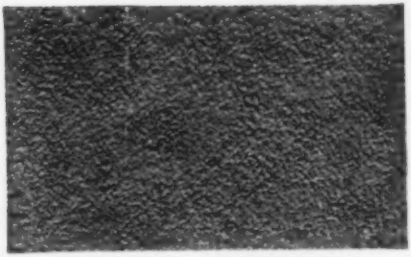

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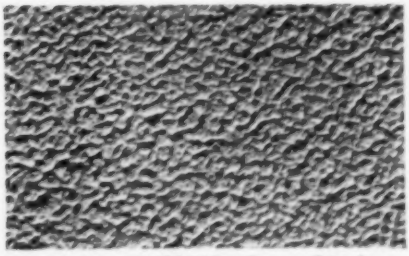

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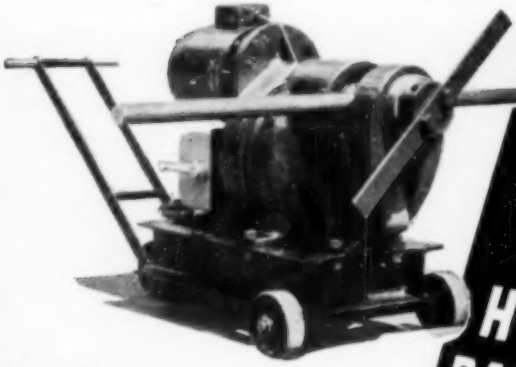

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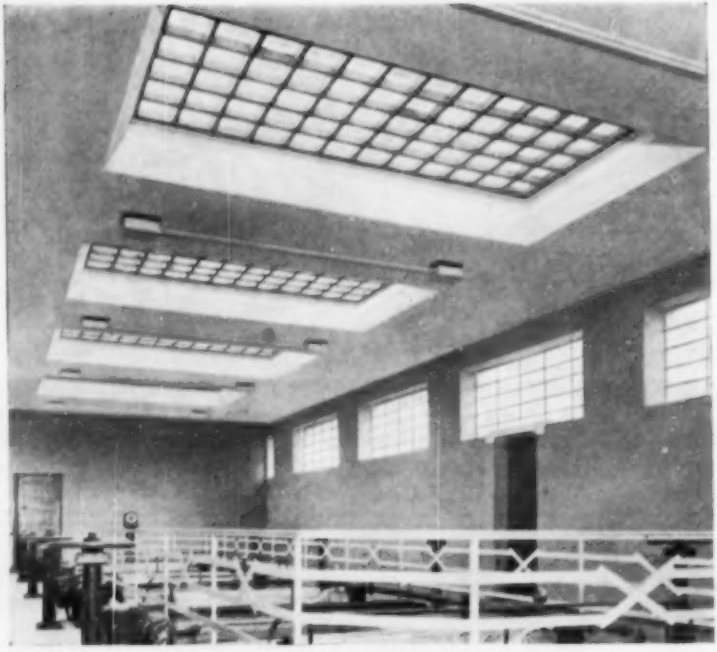

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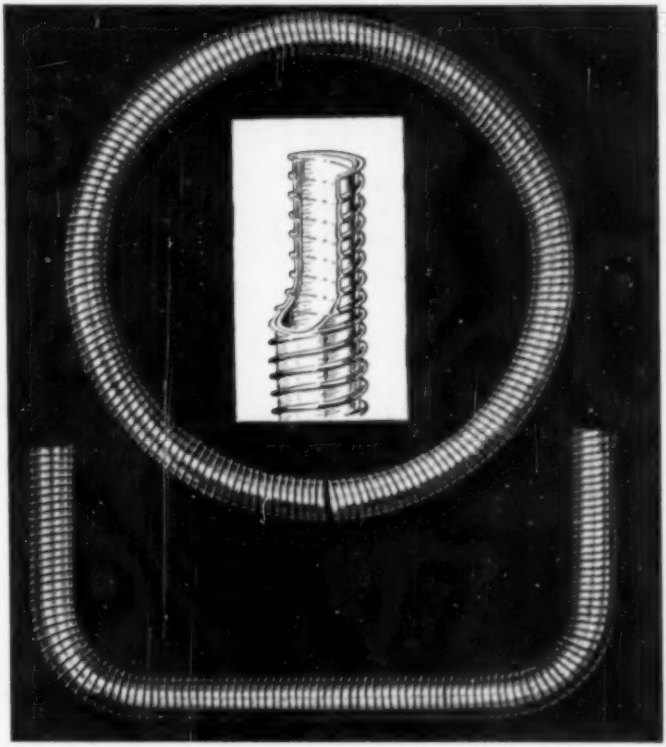

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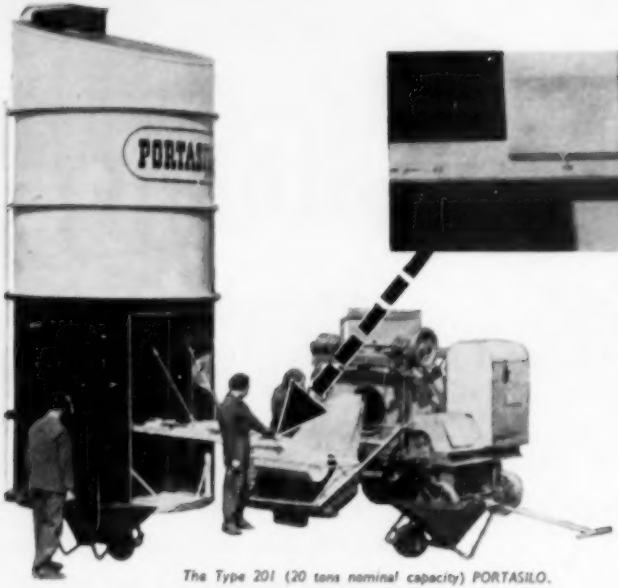

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Volume L, No. 2. LONDON, FEBRUARY, 1955.

EDITORIAL NOTES

The Resistance to Frost of Air-entrained Concrete.

AMONGST the chemicals recently used to reduce the effects of frost on fresh and hardened concrete are resinous air-entraining agents. These chemicals increase the workability of the mixture and allow a smaller water-cement ratio to be used, and this partly or entirely offsets the lower compressive strength caused by their use. What is more important from the point of view of winter concreting is that they increase considerably the resistance of hardened concrete to disintegration caused by repeated cycles of freezing and thawing. For this reason air-entraining agents are used in concrete road slabs to resist the effect of salts applied to the surface to remove ice or to prevent its formation.

An air-entraining agent produces in concrete a very great number of minute air bubbles which are dispersed throughout the mixture. These bubbles act as a lubricant between the particles of aggregate, and their effect is to increase workability. The reason why the presence of these small voids increases the resistance of hardened concrete to freezing and thawing is not, however, so well understood. In mixing ordinary unset concrete the water, when the temperature falls below freezing point, tends to form ice-lenses which have a greater volume than the water of which they are formed. According to a theory propounded by Mr. T. C. Powers and Mr. R. A. Helmut and quoted in a Danish publication on winter concreting,* the tensile strength of ordinary cement paste is sufficient to prevent the formation of ice-lenses, and the water, on expanding as it gets colder, is forced into the concrete. Due to the low permeability of concrete the resulting hydraulic pressure is sufficient to disrupt the concrete, and the disrupting effect is greater still when the water becomes ice. If, however, the concrete contains a great number of closely-spaced air bubbles, as in air-entrained concrete, then the water will be forced into these spaces as it gets colder and the hydraulic pressure thereby relieved. Investigations by one of the authors of this theory have shown that when, due to prolonged frost, the water in the bubbles freezes, the ice thus formed will attract water from adjacent cement-gels without disrupting the bubbles. This removal of the water from the cement gel causes a shrinkage of the concrete, whereas concrete without entrained air expands on freezing.

From this theory a method has been devised of calculating the hydraulic

* "Betonstøbning om Vinteren", by P. Nerenst, E. Rastrap, and G. M. Idorn. (The Danish Institute of Building Research. 1953.)

pressure resulting from the freezing of the water in air-entrained concrete, and by equating this calculated pressure with the tensile strength of the concrete at various ages it is said to be possible to calculate the time during which the temperature of the fresh concrete should be maintained above freezing point. It is claimed that it is therefore possible to determine with some confidence the earliest time when the heating devices and insulation may safely be removed. Although it is not stated in the Danish publication, it would seem reasonable to assume that an explanation of the protective effect of air-entrainment on ice-bound concrete roads treated with thawing salts would be similar, but based upon the growth of crystalline structures in the concrete rather than on the formation of ice-lenses. The somewhat complicated calculations relating the hydraulic pressure to the tensile strength of the cement paste, which include the quantity of water and its viscosity, the saturation-coefficient of the cement paste, the rate of cooling (which depends upon the size and shape of the members), the permeability of the fresh concrete, the average diameter of the air bubbles and their spacing, and the rate of increase of the tensile strength of the cement paste, have been reduced by the Danish authors to the application of graphs and tables. It is doubtful if in a temperate climate the methods would be justified, but engineers and contractors concerned with placing concrete in colder climates might well gain some valuable information from a study of the book, of which an abbreviated translation (without the diagrams and curves) is available in the English language.

Even in a temperate climate the problem of placing concrete and protecting it during the curing period in cold weather is one which has not been satisfactorily solved. In colder climates the problem is of much greater importance and much thought has been given to devising methods of heating the materials before and during mixing and of maintaining the temperature of the concrete above freezing after it has been placed. As well as the use of external sources of heat, advantage has been taken of the heat generated by the chemical actions occurring in the cement during hydration, and to augment this various chemicals have been used which generally have the effect of increasing the rate of hydration of the cement. The total amount of heat generated by different types of Portland cements during setting is very much the same with and without the addition of chemicals, and the temperature of the cement, and consequently of the concrete, is increased for a short period if all the heat is produced in a short time. Thus by increasing the rate of setting the temperature of the mixture is raised initially compared with what it would be with a normal-setting cement, and also the rate of increase of strength is hastened, allowing external sources of heat to be dispensed with much sooner. Generally, however, the added chemicals do not increase the ultimate strength of the concrete nor its resistance to subsequent freezing and thawing of the hardened concrete. The chemical most commonly used is calcium chloride, but some engineers are averse to its use in reinforced concrete until long-period tests have proved that it will not cause corrosion of steel embedded in the concrete in which calcium chloride is included. Nevertheless chemicals are frequently used in order that external sources of heat and means of insulating the concrete are required for a shorter time, as these are the most expensive of the precautions to be taken ; compared with the cost of providing adequate protection in very cold weather, the heating of the materials before and during mixing is relatively inexpensive.

Closed Annular Sections subjected to Bending and Direct Compression.—I.

By J. RYGOL, B.Sc.(Eng.).

THE design of an annular section subjected to bending and compression, using algebraic expressions, is usually done by trial and error and is laborious. The nomographic method presented may be used for either design or analysis. The nomograms are based on algebraic equations and their derivation is shown.

Two cases are considered, namely, (1) Compression throughout the section, and (2) Tension in part of the section. The assumptions made are (a) The symmetrical reinforcement A_s is replaced by a uniform ring of steel of mean radius R, and (b) The concrete is considered to be concentrated at the mean radius R; that is the inside and outside concrete covers are equal. Assumption (a) results in negligible error. Assumption (b) is usually achieved in practice.

Nomograms Nos. 1 to 3 are constructed for the eccentricity e measured from the physical centre of the section. If the thrust N acts at the centre of the section and there is a bending moment M, the equivalent eccentricity of N is $e = \frac{M}{N}$ measured from the centre of the section. In this case the nomograms can be applied directly. When there is tension in part of the section, thrust N, when combined with M, acts at an eccentricity measured from the centroid of the effective section and its position in relation to the centre of section is unknown. An iterative process of solution to meet this latter assumption, using the same nomograms, is adopted. Where there is pure bending, $N = 0$ and $\frac{e}{R} = \infty$.

Compression Throughout the Section (Fig. 1).

The greatest stresses in the concrete are

$$f_{1,2} = \frac{N}{A_{eq.}} \pm \frac{Ne(R + t)}{I_{eq.}} \quad . \quad . \quad . \quad . \quad (1)$$

where $A_{eq.}$ is the equivalent area of concrete, $I_{eq.}$ the equivalent moment of inertia, m the modular ratio, p the ratio of the area of reinforcement to the true area of the concrete, R the mean radius, t half the thickness of the annular ring, N the compressive force, and e the eccentricity.

Let $\frac{t}{R} = a$ and the area of concrete $A = 4\pi Rt$. Then

$$A_{eq.} = 4\pi Rt(1 + mp) = A(1 + mp),$$

and $$I_{eq.} = 2\pi Rt(R^2 + t^2) + 2\pi R^3 tmp = \frac{AR^2}{2}[1 + a^2 + mp].$$

Therefore $$f_{1,2} = \frac{N}{A}\left[\frac{1}{1 + mp} \pm \frac{2e}{R}\left(\frac{1 + a}{1 + a^2 + mp}\right)\right] \quad . \quad . \quad . \quad (2)$$

The ratio $\frac{t}{R} = a$ is small, and an approximation of the right-hand part in the bracket is permissible:

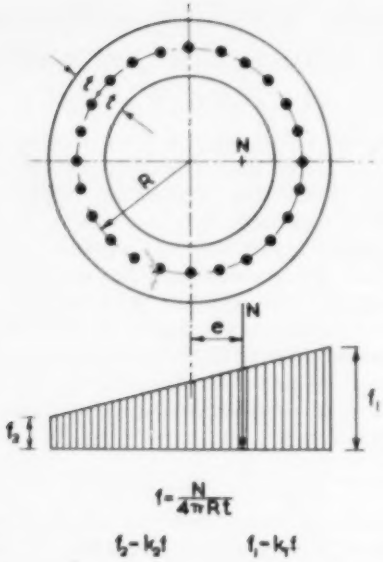


Fig. 1.—Values of k_1 and k_2 from Nomogram No. 1 on pages 82 and 83.

$$\frac{1+a}{1+a^2+mp} \simeq \frac{1+a}{1+mp}\left[1-\frac{a^2}{1+mp}\right] \simeq \frac{1}{1+mp}(1+a-0{\cdot}8a^2).$$

Therefore $$f_{1,2} = \frac{N}{A(1+mp)}\left[1 \pm \frac{2e}{R}(1+a-0{\cdot}8a^2)\right]. \quad . \quad . \quad (2a)$$

The approximation results in an error of less than 0·3 per cent. in the case where $a = 0{\cdot}3$ and $mp = 0{\cdot}9$. Writing $\frac{N}{A} = f$ and $\frac{f_1}{f} = k_1$, $\frac{f_2}{f} = k_2$, equation (2a) becomes

$$f_{1,2} = \frac{f}{1+mp}\left[1 \pm \frac{2e}{R}(1+a-0{\cdot}8a^2)\right], \text{ and}$$

$$k_1 = \frac{1}{1+mp}\left[1+\frac{2e}{R}(1+a-0{\cdot}8a^2)\right]; \quad k_2 = \frac{1}{1+mp}\left[1-\frac{2e}{R}(1+a-0{\cdot}8a^2)\right].$$

Nomogram No. 1 is of the double-intersection type and gives the relationship between $\frac{e}{R}$, $\frac{t}{R}$, mp, k_1, and k_2. The line of zero stress corresponds to $k_2 = 0$, and the limiting value of $\frac{e}{R}$ in that case depends upon the ratio $\frac{t}{R}$; it approaches 0·5 for a very thin annular cross section.

USE OF NOMOGRAM NO. 1.—

(a) Given t, R, p, N, and M, find f_1 and f_2. Decide the value of m and

evaluate $f = \frac{N}{4\pi Rt}$ and $e = \frac{M}{N}$. Draw a vertical line through the known value $\frac{e}{R}$ to intersect the corresponding curve $\frac{t}{R}$. Through the point so obtained draw a horizontal line to intersect a vertical line through mp. Read k_1 and k_2 corresponding to this point. The stresses are $f_1 = k_1 f$ and $f_2 = k_2 f$.

EXAMPLE. $t = 5$ in.; $R = 3$ ft.; $N = 50$ tons; $M = 490$ tons-inches; $p = 2$ per cent.; $m = 18$.

Solution. $f = \frac{N}{4\pi Rt} = \frac{50 \times 2240}{4\pi \times 36 \times 5} = 496$ lb. per square inch.

$$e = \frac{M}{N} = \frac{490}{50} = 9{\cdot}8 \text{ in.} \quad \frac{e}{R} = \frac{9{\cdot}8}{36} = 0{\cdot}272. \quad \frac{t}{R} = \frac{5}{36} = 0{\cdot}139.$$

From Nomogram No. 1, $k_1 = 1{\cdot}18$—therefore $f_1 = 1{\cdot}18 \times 496 = 585$ lb. per square inch. $k_2 = 0{\cdot}29$—therefore $f_2 = 0{\cdot}29 \times 496 = 144$ lb. per square inch.

(b) To design a section for a working stress of f_1 when $f_2 = 0$. Assume a ratio $\frac{t}{R}$. Extend the line $k_2 = 0$ to intersect with the assumed $\frac{t}{R}$-curve. This gives the ratio $\frac{e}{R}$. As e is known $\left(e = \frac{M}{N}\right)$, the mean radii R and t can be calculated. Next evaluate $f = \frac{N}{4\pi Rt}$, and then $k_1 = \frac{f_1}{f}$. A vertical line through the point of intersection of the k_1 and k_2 lines determines the necessary percentage of modular reinforcement mp.

EXAMPLE. $f_1 = 900$ lb. per square inch; $f_2 = 0$; $M = 1500$ tons-inches; $N = 200$ tons.

Solution. $e = \frac{M}{N} = \frac{1500}{200} = 7{\cdot}5$ in. Let $\frac{t}{R} = 0{\cdot}15$. From Nomogram No. 1, $\frac{e}{R} = 0{\cdot}444$.

Therefore $R = 16{\cdot}9$ in. and $t = 0{\cdot}15 \times 16{\cdot}9 = 2{\cdot}63$ in.

$f = \frac{N}{4\pi Rt} = \frac{200 \times 2240}{4\pi \times 16{\cdot}9 \times 2{\cdot}63} = 803$ lb. per square inch, and $k_1 = \frac{f_1}{f} = \frac{900}{803} = 1{\cdot}12$.

From Nomogram No. 1, $mp = 78$ per cent. With $m = 18$, $p = \frac{78}{18} = 4{\cdot}33$ per cent. $A_s = \frac{4{\cdot}33}{100} \times 4\pi \times 16{\cdot}9 \times 2{\cdot}63 = 24{\cdot}2$ sq. in.

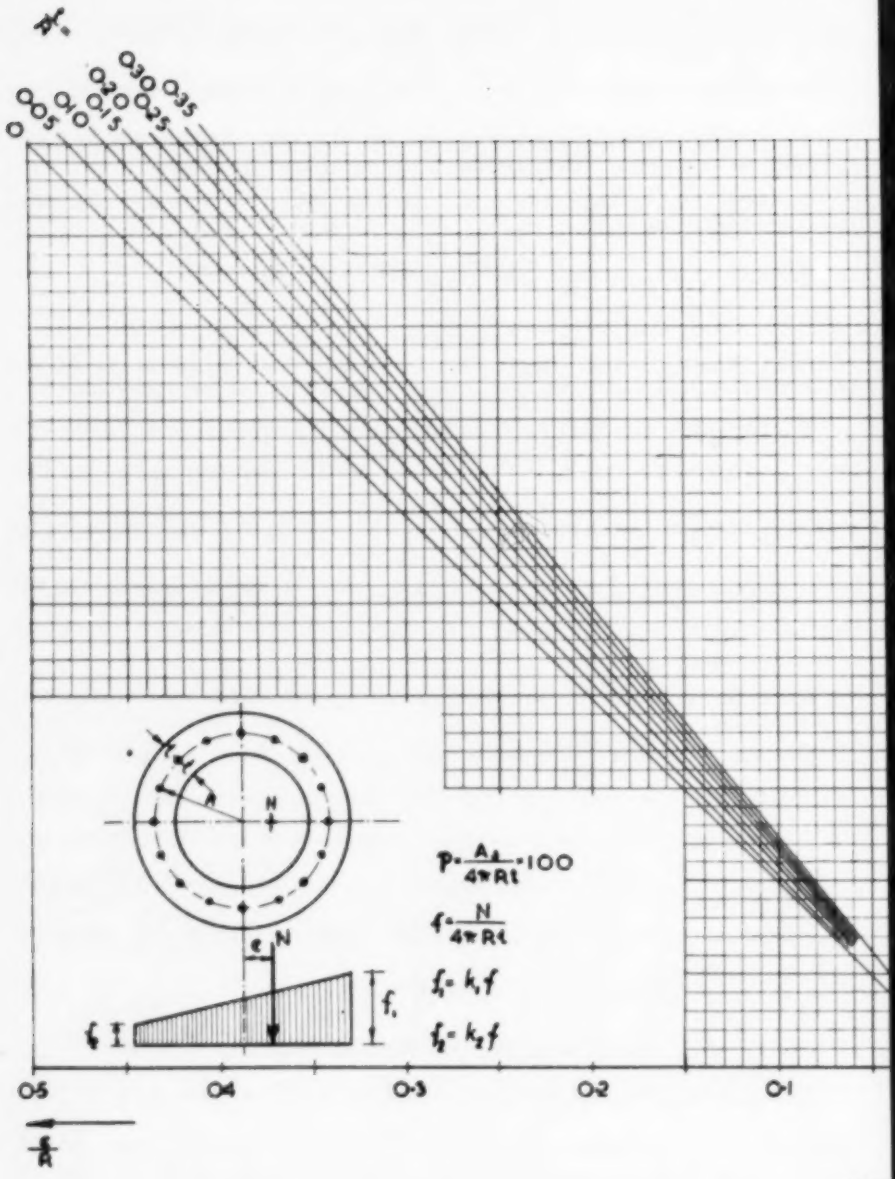
Tension in Part of Section (Fig. 2).

The position of the neutral axis as defined by nR is given by

$$S[e - (1 - n)R] = I \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where S is the statical moment and I the moment of inertia of the transformed concrete section about the neutral axis.

$$S = 4R^2t(1 + mp)(\sin\phi - \phi\cos\phi) + 4R^2tmp[\sin\phi + (\pi - \phi)\cos\phi]$$
$$= 4R^2t[\sin\phi - \phi\cos\phi - \pi mp\cos\phi].$$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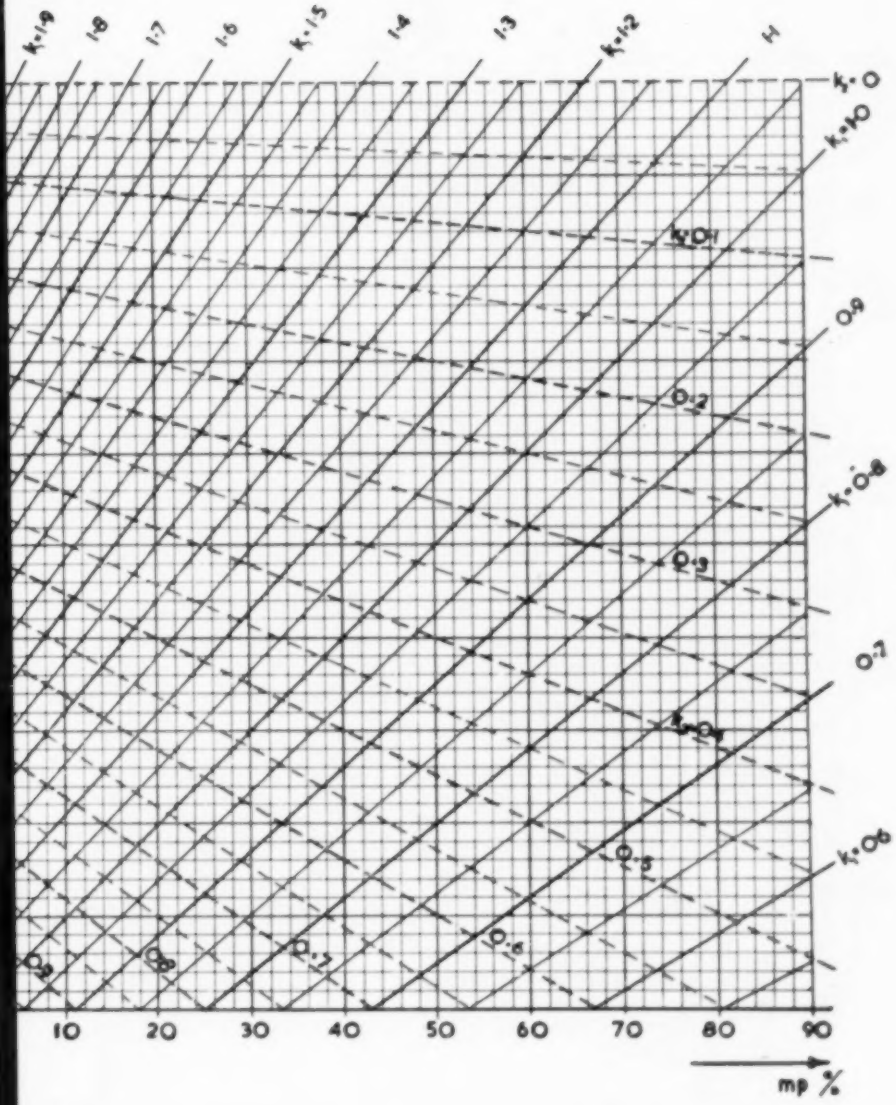
$f_2 = k_2 f$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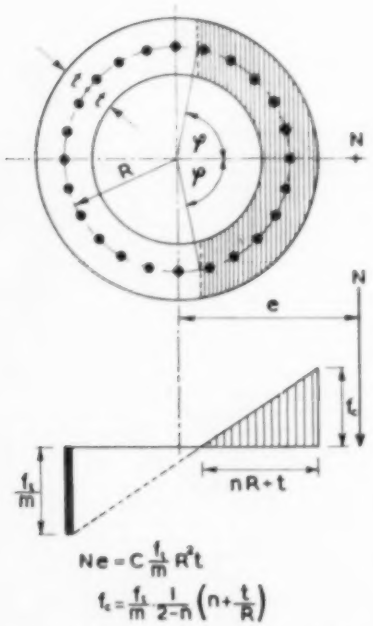


Fig. 2.—Values of C and n from Nomogram No. 2 (page 85).

$$I = 2R^3t(1 + mp)[\phi(1 + 2\cos^2\phi) - 3\sin\phi\cos\phi] + 2R^3tmp[(\pi - \phi)(1 + 2\cos^2\phi) + 3\sin\phi\cos\phi]$$
$$= 2R^3t[(\phi + \pi mp)(1 + 2\cos^2\phi) - 3\sin\phi\cos\phi].$$

Now $n = 1 - \cos\phi$ and, with equation (3) written in the form

$$SR\left[\frac{e}{R} - \cos\phi\right] = I,$$

the expression for $\frac{e}{R}$ becomes

$$\frac{e}{R} = \frac{1}{2}\left[\frac{\pi mp + \phi - \sin\phi\cos\phi}{\sin\phi - \phi\cos\phi - \pi mp\cos\phi}\right] \quad . \quad . \quad . \quad . \quad . \quad (4)$$

$$= \frac{1}{2}\left[\frac{\pi mp + arc\cos(1-n) - (1-n)\sqrt{n(2-n)}}{\sqrt{n(2-n)} - (1-n)arc\cos(1-n) - (1-n)\pi mp}\right] \quad . \quad (4a)$$

Nomogram No. 2 is based on (4a) and gives the position of the neutral axis n as a function of $\frac{e}{R}$ and mp.

To ascertain the stresses in the concrete, the compressive force N must be equal to the total internal forces. Let F_t be the total tensile force in the steel,

F_c the total compressive force in the steel, and P the total compressive force in the concrete, then

$$N = F_c + P - F_t \quad . \quad . \quad . \quad . \quad . \quad (5)$$

$F_t = 4R^2tkmp[\sin\phi + (\pi - \phi)\cos\phi]$, $F_c = 4R^2tkmp[\sin\phi - \phi\cos\phi]$, and $P = 4R^2tk[\sin\phi - \phi\cos\phi]$, where k is the stress in the concrete at unit distance from the neutral axis. Substituting in equation (5),

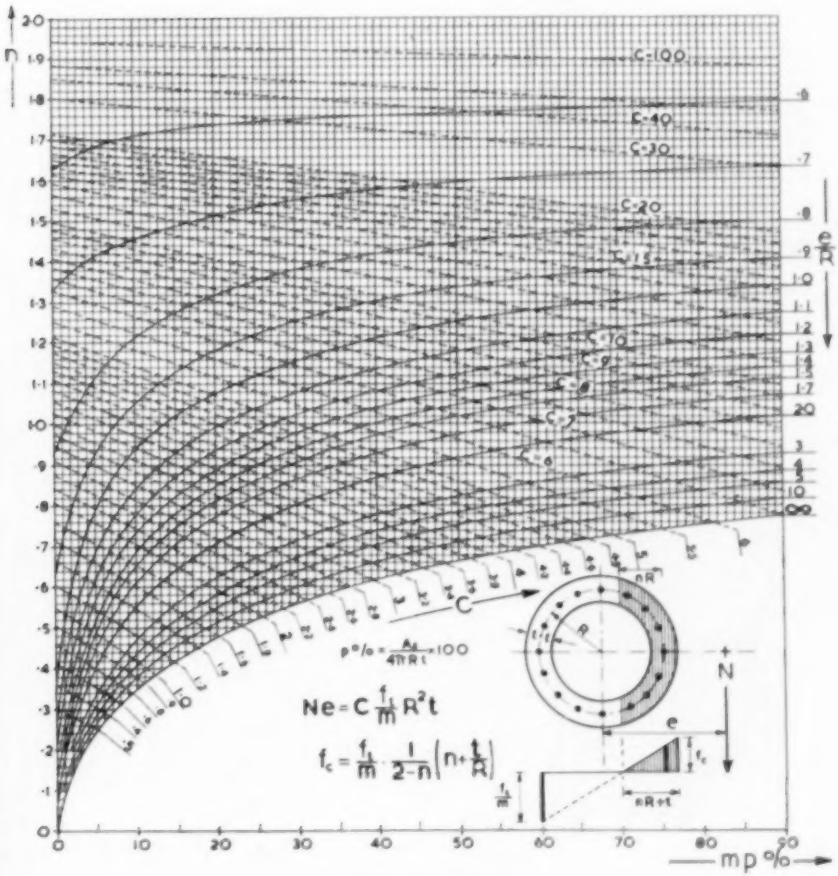
$$k = \frac{N}{4R^2t[\sin\phi - (\phi + \pi mp)\cos\phi]} \quad . \quad . \quad . \quad (6)$$

and

$$\frac{f_t}{m} = (1 + \cos\phi)Rk = \frac{N(1 + \cos\phi)}{4Rt[\sin\phi - \phi\cos\phi - \pi mp\cos\phi]} \quad . \quad (7)$$

But

$$\frac{e}{R} = \frac{1}{2}\left[\frac{\pi mp + \phi - \sin\phi\cos\phi}{\sin\phi - \phi\cos\phi - \pi mp\cos\phi}\right] \quad . \quad . \quad . \quad (4)$$



Nomogram No. 2.

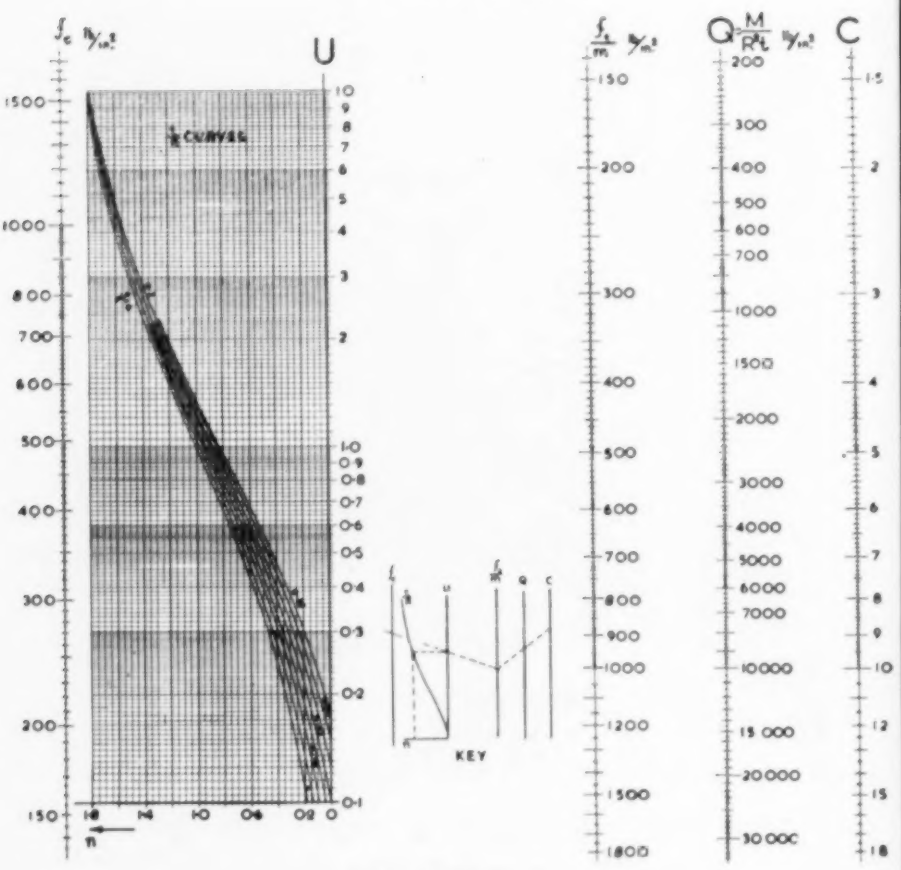
Therefore $\frac{f_t}{m} = \frac{Ne}{R^2 t}\left[\frac{1 + \cos\phi}{2(\pi mp + \phi - \sin\phi\cos\phi)}\right]$. With $Ne = M$, and writing $\frac{M}{R^2 t} = Q$, this becomes

$$\frac{Q}{\frac{f_t}{m}} = \frac{2(\pi mp + \phi - \sin\phi\cos\phi)}{1 + \cos\phi} = C \quad . \quad . \quad . \quad (8)$$

where C is a dimensionless factor. Curves for C expressed as a function of n and mp are drawn in Nomogram No. 2.

The maximum stress in the concrete is given by

$$f_c = \frac{f_t}{m}\left[\frac{R(1 + \cos\phi) + t}{R(1 + \cos\phi)}\right] = \frac{f_t}{m}U \quad . \quad . \quad . \quad (9)$$



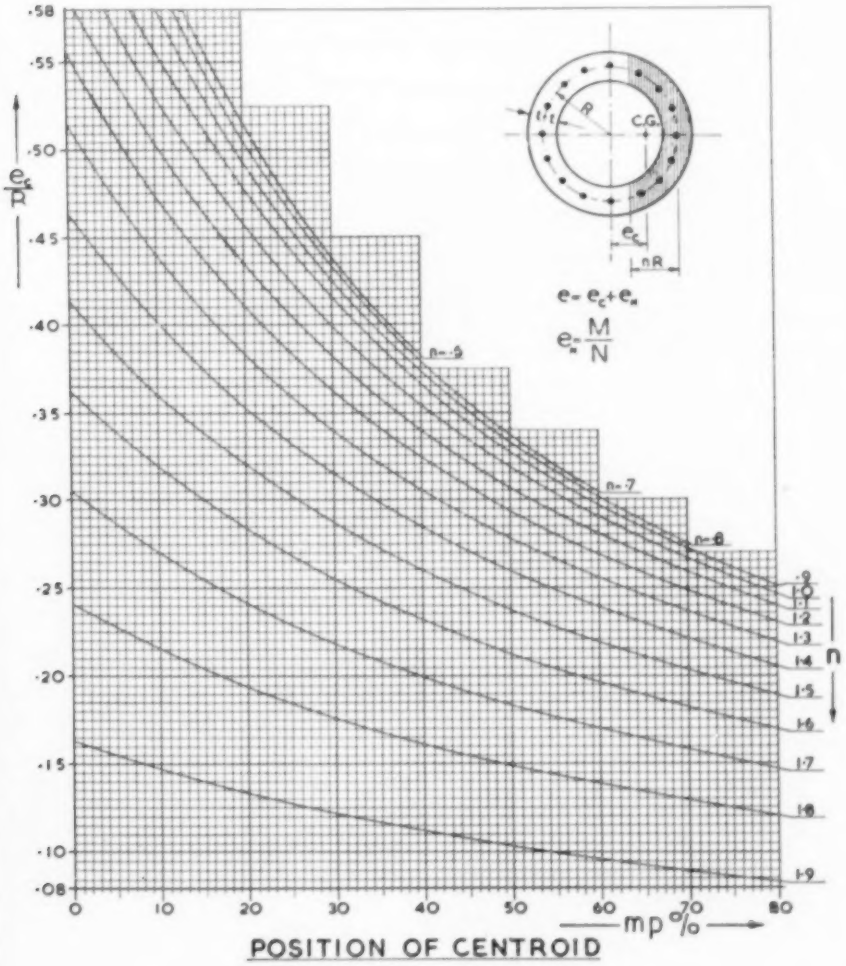
Nomogram No. 3.

where U is the ratio of the stress in the concrete to the modular tensile stress in the steel.

Transforming equation (9), $U = \frac{n}{2-n} + \frac{t}{R}\left(\frac{1}{2-n}\right)$ . . . (9a)

Nomogram No. 3 gives the relationship between U, n, Q, C, $\frac{t}{R}$, f_c, and $\frac{f_t}{m}$.

Lines C, Q, $\frac{f_t}{m}$ form one part of the nomogram and $\frac{f_t}{m}$, U, f_c the other. The $\frac{f_t}{m}$



Nomogram No. 4. $\left(p\,\% = \frac{A_s}{4\pi Rt}\,100.\right)$

line is therefore a transfer line. The value U in relation to n and $\frac{t}{R}$ can be read from the intersecting diagram.

When the permissible stress in the concrete is reached first, then C in (8) must be expressed in terms of f_c instead of $\frac{f_t}{m}$. Thus

$$C = \frac{Q}{f_c}\left(\frac{n + \frac{t}{R}}{2 - n}\right) \quad . \quad . \quad . \quad . \quad . \quad (10)$$

When the eccentricity of thrust N given by the ratio of M to N is to be measured from the centroid of the effective section the following method can be used. Let $e_N = \frac{M}{N}$ and e_c be the distance of the centroid of the transformed concrete section measured from the centre of the annular ring. The eccentricity of N measured from the centre of the section is then

$$e = e_c + e_N \quad . \quad . \quad . \quad . \quad . \quad (11)$$

Now

$$\frac{e_c}{R} = \frac{\sqrt{n(2-n)}}{arc\cos(1-n) + \pi mp} \quad . \quad . \quad . \quad . \quad (12)$$

Nomogram No. 4 is in accordance with (12) and gives the ratio $\frac{e_c}{R}$ in terms of n and mp.

The position of n obtained with the assumption $e = e_N$ is used for the first approximation of n. With the value of n so obtained, find from Nomogram No. 4 the position of the centroid e_c, evaluate $e = e_N + e_c$, and for this value of e find from Nomogram No. 2 the new position of n. With the new n find e_c and repeat this procedure until a satisfactory result is obtained.

(To be continued.)

Piles Formed in Paper Tubes.

PAPER (or millboard) tubes were used to form more than 200 piles about 15 ft. long for a warehouse at Lancaster, South Carolina, U.S.A. The site comprised loose filling on rock at a depth of 15 ft., and the water table was 4 ft. from the surface. A boring machine with an auger of 20 in. diameter was used, and a paper tube of 16 in. diameter immediately dropped into the hole. Water rose rapidly in the hole, but it is stated that there was little difficulty in pumping out the water and inserting the prefabricated reinforcement and placing the concrete. The bottoms of the tubes were liable to disintegrate if too long a period elapsed between placing the tubes and concreting, and this was overcome by painting with bitumen the bottom 4 ft. or 5 ft. of the tube, or in some cases encasing the lower part of the tube in a thin metal collar. In a few cases where the mud could not be kept out of the holes long enough to allow the reinforcement and concrete to be placed, 10-gauge metal tubes were dropped into the holes and driven to the rock by a pile-hammer. The method was devised by the engineering department of the owners of the building, the Spring Mills Cotton Mills.

A School in South-east London.

THE school shown in the illustrations has been built at Kidbrooke for the London County Council to accommodate 2000 girls. The classroom blocks, mostly three-story buildings, are of reinforced concrete slab, beam, and column construction, with a shell roof of 56 ft. span over the kitchen and north-light shell roofs continuous over two spans of 40 ft. each for the craft-rooms.

The main assembly hall has a concrete dome 3 in. thick which is rectangular on plan (*Figs.* 2 to 4). The upper parts of the walls are 8 in. thick and are monolithic with the dome; they are supported by a column at each corner. The walls on the longer sides have large window openings (*Fig.* 2); from the underside of these walls (which are designed as beams) are suspended the inner sides of the flat roofs of the galleries at each side of the hall. Where the windows occur the oval concrete mullions, designed as hangers, transfer to the walls the load from one edge of the flat roofs. The walls were prestressed so that they act as beams to carry the vertical loads and as ties to resist the forces tending to spread the dome. The dome is almost free from bending moments because the prestressing of the walls almost eliminates strains at the edges of the dome relative to the walls.

Fig. 1.—Class Room Buildings.

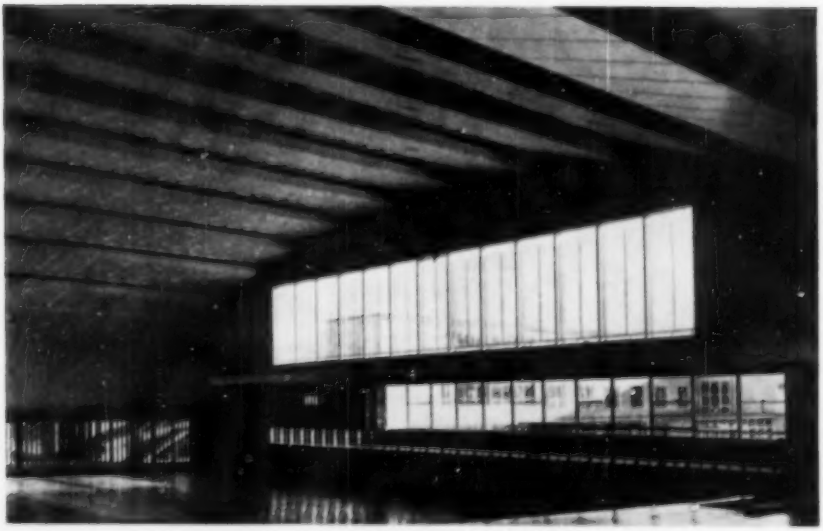


Fig. 2.—The Assembly Hall from the Stage.

Fig. 3.—View of Assembly Hall.

An acoustic ceiling inside the hall is suspended from the dome by steel rods; the space between the ceiling and the dome contains walkways, lighting equipment, ventilation ducts, and other services. Rainwater is drained from the dome by channels near its edges. Continuity of the dome and walls is obtained by haunches on the inside of the roof which are large enough to incorporate the drainage channels.

The architects are Messrs. Slater, Uren & Pike, and the consulting structural engineers Messrs. Ove Arup & Partners. The main contractors were Messrs. Gee, Walker & Slater, Ltd., and the contractors for the foundations Messrs. J. Garrett & Sons, Ltd.

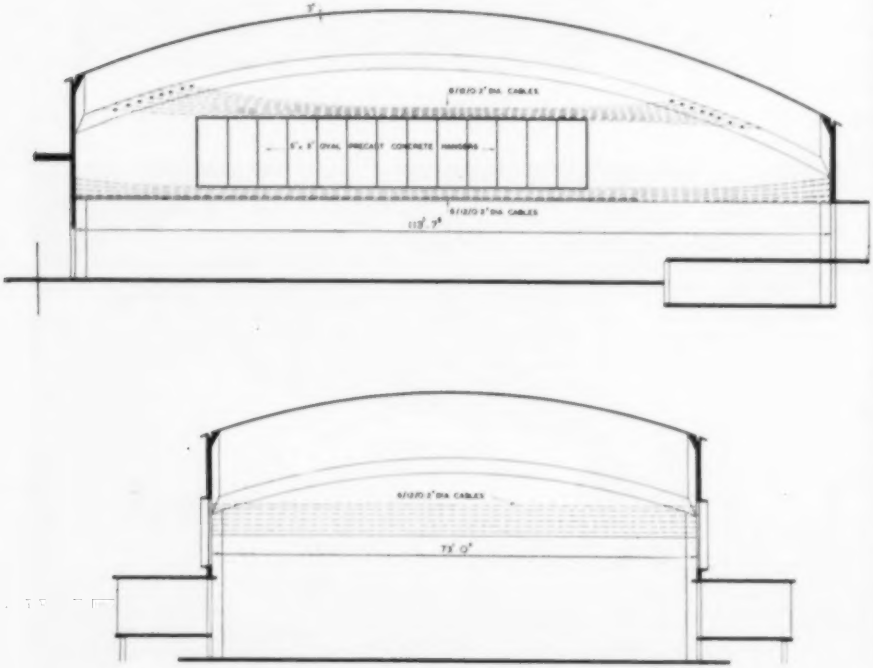


Fig. 4.—Cross Sections of Assembly Hall.

Conveyor Gantries at a Gasworks.

EXTENSIONS now in progress at Willoughby Lane (London) gas works for the Eastern Gas Board, Tottenham Division, include coal and coke handling plant to serve a new carbonizing plant. An extensive system of conveyors (*Fig.* 1) was required to convey the coal from the wagon tipplers to the carbonizing plant or to the store, to move coal from the store to the carbonizing plant, and to carry coke from the carbonizing plant to the bunkers. Most of the overhead conveyors are carried in prestressed gantries having a total length of 2927 ft. The transfer towers, coal hoppers, and coke hoppers are constructed in reinforced concrete.

The gantries (*Figs.* 2, 3, and 4) are 10 ft. 2 in. deep and vary in width from 5 ft. 8 in. to 11 ft. 6 in. The sides are prestressed girders 6 ft. 6 in. deep, with vertical ribs which continue as columns to the roof. The minimum thickness of the web between the ribs is 7 in. and the end panels are 12 in. thick. Concrete purlins, fixed to the roof frame, carry a covering of curved asbestos-cement sheeting. The floor is a reinforced concrete slab spanning between the girders.

All the gantries, except that over the railway, are made up of simply-supported spans, in most cases 90 ft. long. Over the railway the gantry is continuous over two spans of 108 ft. and 78 ft. and is designed according to the method proposed by M. Y. Guyon. The simply-supported spans are designed by Professor Magnel's method.

The gantries are supported at the ends

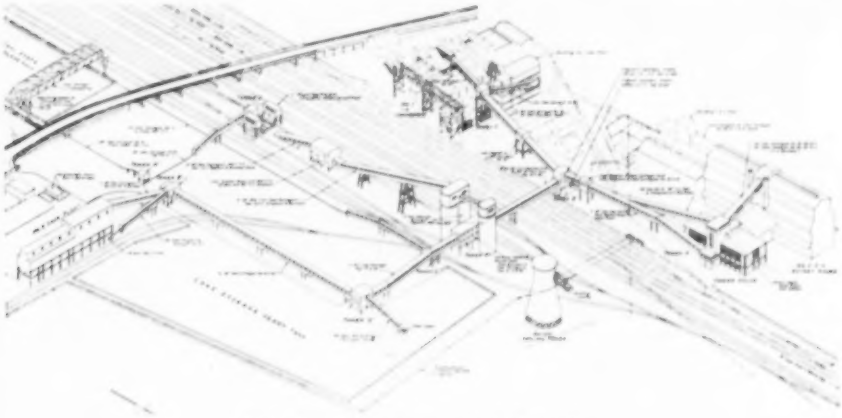


Fig. 1.—Arrangement of Gantries.

Fig. 2.—Completed Gantries.

Fig. 3.—Interior of Conveyor Gantry.

Fig. 4.—A Gantry During Construction.

on transfer towers and intermediately on trestles, double trestles being provided to allow for the stressing jacks. All the spans have a fixed support at one end and a sliding bearing at the other. Between the spans a gap of about 1 in. is provided to allow for longitudinal shrinkage and thermal movements.

The girders were prestressed by the Magnel-Blaton system. Three cables, each of twenty-four 0·276-in. diameter wires, were used for spans up to 90 ft. and four similar cables for the span of 108 ft. The cables are curved to a minimum radius of about 370 ft. except that in the two continuous spans over the railway the radius over the intermediate support is 100 ft. The wire used in the first part of the work had an ultimate strength of 90 to 100 tons per square inch and was tensioned to 54 tons per square inch. For most of the work, however, wire with an ultimate strength of 95 to 105 tons per square inch was used, and tensioned to 57 tons per square inch.

The design required that the compressive stress in the concrete when the stress was transferred to the girders should not exceed 2500 lb. per square inch and that there should be no tensile stress in the concrete. At full working load, after allowances for creep and shrinkage, the compressive stress in the concrete was not to exceed 2000 lb. per square inch. A minimum residual compressive stress of 100 lb. per square inch was required in all sections as a precaution against tensile cracks. In a 90-ft. span the actual maximum compressive stress in the bottom flange due to prestressing is 1403 lb. per square inch and under working load the compressive stress in the top flange is 1274 lb. per square inch. The continuous gantry over the railway has a maximum compressive stress of 1790 lb. per square inch due to prestressing, and 1880 lb. per square inch under working load.

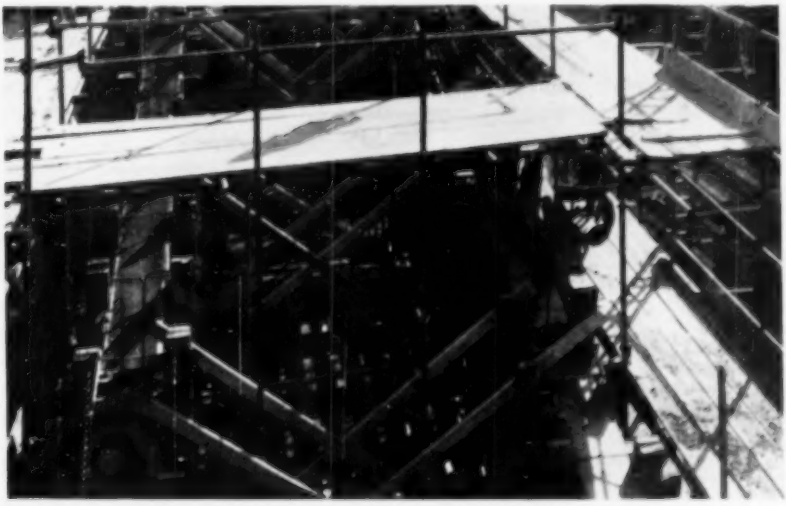


Fig. 5.—Shuttering for Girders.

The cables are in rubber ducts 2½ in. by 3½ in. in cross section stiffened by a ⅞-in. diameter steel rod. The ducts were held in position by ½-in. diameter mild steel stirrups and supported at the required curvature by steel rods passing through the shutters. The cables in the simply-supported girders were tensioned by an hydraulic jack at one end only, while a jack at each end was used for tensioning the cables on the two-span continuous gantry. After the wires were tensioned, the ducts were filled with grout comprising 1 part of cement to 1½ parts of sand

applied at a pressure of 25 lb. per square inch.

The mixture used for the concrete was 1 cwt. of cement, 170 lb. of sand, 290 lb. of ¾-in. to ⅜-in. shingle, and not more than 55 lb. of water, and the specified slump was ¾ in. The specified minimum crushing strength was 6000 lb. per square inch at 28 days. The concrete was proportioned by weight and compacted by internal vibrators. Test cubes cured under the same conditions as the concrete in the work had compressive strengths of 6000 to 8000 lb. per square inch at 14 days and between 7000 and 9000 lb. per square inch at 28 days. Some cubes made when the temperature was between 33 deg. and 40 deg. Fahr. for about two weeks had compressive strengths of 5300 lb. per square inch at 14 days and 7700 lb. at 28 days. *Fig.* 5 shows the shuttering used for the girders.

Other Structures.

In addition to the coal hoppers and gantry transfer towers, other construction has been carried out in reinforced concrete, including 36 elevated purifier boxes each 30 ft. square by 7 ft. 6 in. deep, coke hoppers, paved storage areas for coal and coke, a frame for a power house, and various foundations. The purifier boxes were designed in accordance with the Code of Practice for the Storage of Liquids, and a 1 : 1½ : 3 mixture was used. For the other reinforced concrete work a 1 : 2 : 4 mixture was used.

Air-entrained Concrete.

For most of the construction air-entrained concrete was used. Tests were made when a paved coke-yard was under construction, using air-entrained concrete and ordinary concrete made with similar materials and under similar conditions and mixed in batches containing 1 cwt. of cement. With the addition of the air-entraining agent less water was required for the same workability with the result that the air-entrained concrete had greater strength.

The air-entrained concrete contained, per cubic yard, 499·8 lb. of cement, 1116 lb. of sand, and 2232 lb. of gravel, and the weight of the concrete as it left the mixer was 142·2 lb. per cubic foot. After storage for 28 days, 6-in. test cubes had an average density of 144·8 lb. per cubic foot and a compressive strength of 6600 lb. per square inch.

The ordinary concrete contained, per cubic yard, 493 lb. of cement, 1214 lb. of sand, and 2203 lb. of gravel. After storage for 28 days, 6-in. test cubes had a density of 146·2 lb. per cubic foot and a compressive strength of 4668 lb. per square inch.

The greater density of the test cubes made with both types of concrete is no doubt due to extra compaction, and the greater strength of the air-entrained concrete is due to the lower water-cement ratio.

The Divisional Engineer of the Eastern Gas Board is Mr. E. O. Rose, B.Sc.(Eng.), M.I.C.E., and the constructional engineer Mr. W. T. Gedge, M.I.Gas.E. The contractors and associate designers for the concrete work were Messrs. Peter Lind & Co., Ltd.

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Analysis of Statically-indeterminate Structures by the Deformation Method.—VI.*

By M. SMOLIRA, Ph.D., A.M.Inst.C.E., D.I.C.

Vierendeel Trusses.

In this article Vierendeel trusses are analysed, that is, trusses composed of rectangular or trapezoidal panels without diagonal members, with the end connections of all members rigid, and designed to resist bending moments. Deformations due to axial forces are generally neglected.

In the case of symmetrical trusses, with loads only at the joints, bending moments and shearing forces on the upper and lower chords are equal, and the points of contraflexure of vertical members are at mid-height. The equations of equilibrium are therefore set out for one chord only. In the case of unsymmetrical trusses the points of contraflexure of vertical members are not at mid-height and, generally, it is necessary to set out equations of equilibrium for the joints at the upper and lower chords. This, however, may be avoided by assuming that the bending moments on each corresponding upper and lower member are in the ratio of the stiffness of the members. This is illustrated by a numerical example. Vierendeel trusses with unsymmetrical or inclined chords and continuous Vierendeel trusses are also discussed, and numerical examples illustrate the procedure. When bending moments are calculated from equations of equilibrium, all shearing and axial forces are calculated from conditions of statics.

Two-bay Vierendeel Trusses.—Assume that joints A, B, C, and E (*Fig.* 55)

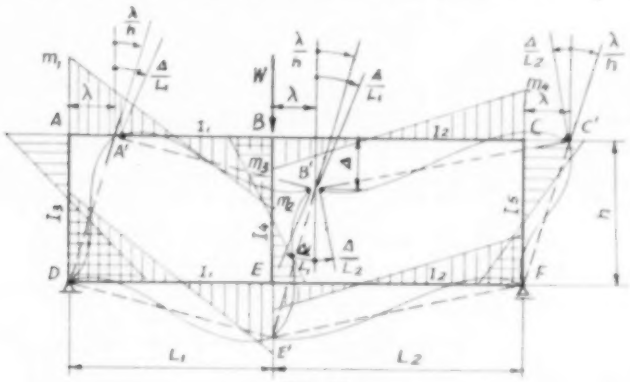

Fig. 55.

are at their final positions of A', B', C', and E', λ being the horizontal sway and Δ the vertical deflection of joint B. The deformed shape of the truss and the angular gaps are shown in *Fig.* 55. Statically-indeterminate bending moments m_1, m_2, m_3, m_4, and deflections λ and Δ, are calculated from four conditions of equilibrium of angular deformations and from two conditions of shear as in equation (74). The third equation is set out for members between joints ABE,

* Previous articles appeared in this journal for July, August, September, October, and November, 1954.

$$
\left.\begin{aligned}
&\text{DAB}\ldots m_1\alpha_{ab}+m_1\alpha_{ad}-m_1\beta_{ad}-m_2\beta_{ab}=\frac{\Delta}{L_1}-\frac{\lambda}{h}\\
&\text{ABC}\ldots m_2\alpha_{bd}+m_3\alpha_{bc}-m_1\beta_{ba}-m_4\beta_{bc}=\frac{\Delta}{L_1}+\frac{\Delta}{L_2}\\
&\text{ABE}\ldots m_2\alpha_{be}+(m_2-m_3)\alpha_{be}\ -m_1\beta_{ba}-(m_2-m_3)\beta_{be}=\frac{\Delta}{L_1}-\frac{\lambda}{h}\\
&\text{BCF}\ldots m_4\alpha_{cb}+m_4\alpha_{cf}-m_3\beta_{cb}-m_4\beta_{cf}=\frac{\Delta}{L_2}+\frac{\lambda}{h}\\
&\text{S.c.BE}\ldots\frac{m_1+m_2}{L_1}+\frac{m_3+m_4}{L_2}=\frac{W}{2}\\
&\text{S.c ABC}\ldots\frac{m_1}{h}+\frac{m_2-m_3}{h}=\frac{m_4}{h}
\end{aligned}\right\}\quad\ldots\quad(74)
$$

$$
\left.\begin{aligned}
&\text{DAB}\ldots m_1\frac{L_1}{3I_1}+m_1\frac{h}{3I_3}-m_1\frac{h}{6I_3}-m_2\frac{L_1}{6I_1}=\frac{\Delta}{L_1}E-\frac{\lambda}{h}E\\
&\text{ABC}\ldots m_2\frac{L_1}{3I_1}+m_3\frac{L_2}{3I_2}-m_1\frac{L_1}{6I_1}-m_4\frac{L_2}{6I_2}=\frac{\Delta}{L_1}E+\frac{\Delta}{L_2}E\\
&\text{ABE}\ldots m_2\frac{L_1}{3I_1}+(m_2-m_3)\frac{h}{3I_4}-(m_2-m_3)\frac{h}{6I_4}-m_1\frac{L_1}{6I_1}=\frac{\Delta}{L_1}E-\frac{\lambda}{h}E\\
&\text{BCF}\ldots m_4\frac{L_2}{3I_2}+m_4\frac{h}{3I_5}-m_4\frac{h}{6I_5}-m_3\frac{L_2}{6I_2}=\frac{\Delta}{L_2}E+\frac{\lambda}{h}E\\
&\text{S.c BE}\ldots\frac{m_1+m_2}{L_1}+\frac{m_3+m_4}{L_2}=\frac{W}{2}\\
&\text{S.c ABC}\ldots\frac{m_1}{h}+\frac{m_2-m_3}{h}=\frac{m_4}{h}
\end{aligned}\right\}\quad\ldots\quad(75)
$$

$$
\left.\begin{aligned}
m_1&=\frac{WL}{2}\cdot\frac{3L}{6L+h}\\
m_2&=\frac{WL}{2}\cdot\frac{3L+h}{6L+h}
\end{aligned}\right\}\quad\ldots\quad(76)
$$

but it may be replaced by a similar equation for members between joints EBC.

If all members are prismatic, equations (74) reduce to (75), and if all members have the same cross section and length the solution of (75) may be reduced to (76).

EXAMPLE.—For the two-bay truss in *Fig.* 56, and from equations (75), we get (76*a*).

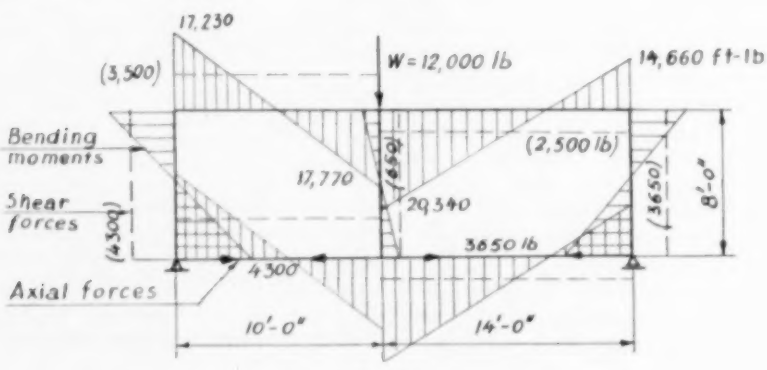

Fig. 56.

$$
\left.\begin{aligned}
&DAB\ldots \tfrac{10}{3}m_1 + \tfrac{8}{3}m_1 + \tfrac{8}{6}m_1 - \tfrac{10}{6}m_2 = \tfrac{\Delta}{L_1}EI - \tfrac{\lambda}{h}EI\\
&ABC\ldots \tfrac{10}{3}m_2 + \tfrac{14}{3}m_2 - \tfrac{10}{6}m_1 - \tfrac{14}{6}m_4 = \tfrac{\Delta}{L_1}EI + \tfrac{\Delta}{L_2}EI\\
&ABE\ldots \tfrac{10}{3}m_2 + (m_2 - m_3)\tfrac{8}{3} - (m_2 - m_3)\tfrac{8}{6} - \tfrac{10}{6}m_1 = \tfrac{\Delta}{L_1}EI - \tfrac{\lambda}{h}EI\\
&BCF\ldots \tfrac{14}{3}m_4 + \tfrac{8}{3}m_4 - \tfrac{8}{6}m_4 - \tfrac{14}{6}m_3 = \tfrac{\Delta}{L_2}EI + \tfrac{\lambda}{h}EI\\
&ScBE\ldots \tfrac{m_1+m_2}{10} + \tfrac{m_3+m_4}{14} = \tfrac{1}{2}\times 12{,}000\\
&ScABC\ldots m_1 + m_2 - m_3 = m_4
\end{aligned}\right\} \quad (76a)
$$

from which $m_1 = 17{,}230$ ft.-lb., $m_2 = 17{,}770$ ft.-lb., $m_3 = 20{,}340$ ft.-lb., and $m_4 = 14{,}660$ ft.-lb. *Fig.* 56 shows the distribution of these bending moments, the shearing forces, and the axial forces.

Four-bay Vierendeel Trusses.—Multiple-bay Vierendeel trusses are analysed by the method used for two-bay trusses. In four-bay trusses with a uniformly-distributed load placed symmetrically on the top and bottom chords (*Fig.* 57) the angular gaps at each joint are influenced by the translations Δ_1 and Δ_2 of joints B and C and also by angular deformations θ due to the external system of loading between the joints. Due to geometrical symmetry of the truss and the load there is no sway, and the equations of equilibrium are as in (77), from which (78) are obtained.

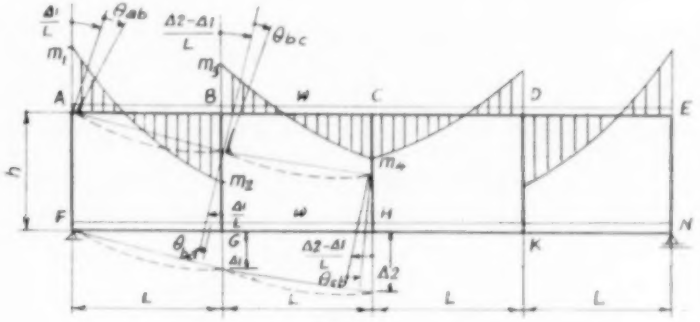

Fig. 57.

$$
\left.\begin{aligned}
&FAB\ldots m_1\tfrac{L}{3} + m_1\tfrac{h}{3} + m_1\tfrac{h}{6} - m_2\tfrac{L}{6} = \tfrac{wL^3}{24} + \tfrac{\Delta^1}{L}EI\\
&ABG\ldots m_2\tfrac{L}{3} + (m_2+m_3)\tfrac{h}{3} - (m_2+m_3)\tfrac{h}{6} - m_1\tfrac{L}{6} = \tfrac{\Delta^1}{L}EI - \tfrac{wL^3}{24}\\
&GBC\ldots m_3\tfrac{L}{3} + (m_2+m_3)\tfrac{h}{3} - (m_2+m_3)\tfrac{h}{6} - m_4\tfrac{L}{6} = \tfrac{\Delta 2-\Delta 1}{L}EI + \tfrac{wL^3}{24}\\
&BCD\ldots m_4\tfrac{L}{3} - m_3\tfrac{L}{6} = \tfrac{\Delta 2-\Delta 1}{L}EI - \tfrac{wL^3}{24}\\
&ScBG\ldots \tfrac{m_1+m_2}{L} - \tfrac{m_3+m_4}{L} = wL\\
&ScCH\ldots \tfrac{m_3+m_4}{L} = \tfrac{1}{2}wL
\end{aligned}\right\} \quad (77)
$$

$$\left.\begin{aligned} m_1 &= wL^3 \cdot \frac{33L+16h}{36L^2+18hL+h^2} \\ m_2 &= \frac{wL^2}{2} \cdot \frac{42L^2+22hL+3h^2}{36L^2+18hL+h^2} \\ m_3 &= \frac{wL^4}{6L+h} \cdot \frac{33L+16h}{36L^2+18hL+h^2} \\ m_4 &= \frac{wL^2}{2} - m_3 \end{aligned}\right\} \quad \ldots \quad (78)$$

$$\left.\begin{aligned} &FAB\ldots\ m_1\frac{L_1}{3I_1}+m_1\frac{h}{3I_3}-m_1\frac{h}{6I_3}-m_2\frac{L_1}{6I_1}=\frac{\Delta_1}{L_1}E \\ &ABG\ldots\ m_2\frac{L_1}{3I_1}+\left(m_2+m_3\right)\frac{h}{3I_4}-\left(m_2+m_3\right)\frac{h}{6I_4}-\frac{L_1}{6I_1}m_1=\frac{\Delta_1}{L_1}E \\ &GBC\ldots\ m_3\frac{L_2}{3I_2}+\left(m_2+m_3\right)\frac{h}{3I_4}-\left(m_2+m_3\right)\frac{h}{6I_4}-m_4\frac{L_2}{6I_2}=\frac{\Delta_2-\Delta_1}{L_2}E \\ &BCD\ldots\ m_4\frac{L_2}{3I_2}-m_3\frac{L_2}{6I_2}=\frac{\Delta_2-\Delta_1}{L_2}E \\ &Sc\,BG\ldots\ \frac{m_1+m_2}{L_1}-\frac{m_3+m_4}{L_2}=\frac{1}{2}W_1 \\ &Sc\,CH\ldots\ \frac{m_3+m_4}{L_2}=\frac{1}{4}W_2 \end{aligned}\right\} \quad \ldots \quad (79)$$

$$\left.\begin{aligned} m_1 &= \frac{3L^2\left[3(2L+h)W_1+(3L+2h)W_2\right]}{2(36L^2+18hL+h^2)} \\ m_2 &= \frac{L}{4}\cdot(2W_1+W_2)-m_1 \\ m_3 &= \frac{3L+h}{h}\cdot(m_1-m_2) \\ m_4 &= \frac{1}{4}W_2L-m_3 \end{aligned}\right\} \quad \ldots \quad (80)$$

For a four-bay symmetrical truss with a symmetrical system of loading (*Fig.* 58), equations (77) require only slight alteration as in (79), from which (80) are obtained.

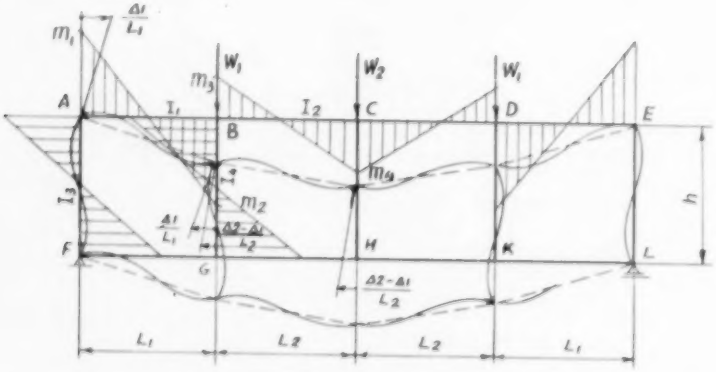

Fig. 58.

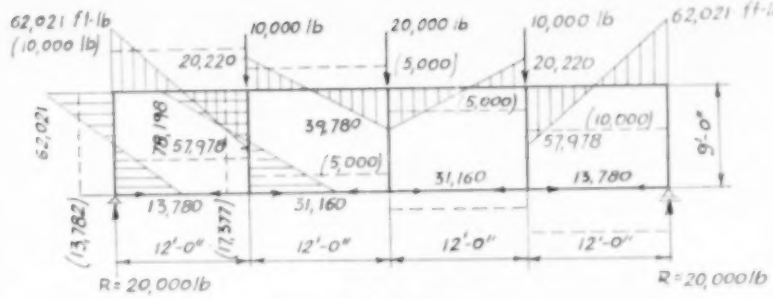

Fig. 59.

$$
\left.
\begin{aligned}
&\text{FAB} \quad \tfrac{12}{3} m_1 + \tfrac{9}{3} m_1 - \tfrac{9}{6} m_1 - \tfrac{12}{6} m_2 = \tfrac{\Delta_1}{L} EI \\
&\text{ABG} \quad \tfrac{12}{3} m_2 + \tfrac{9}{3}(m_2+m_3) - \tfrac{9}{6}(m_2+m_3) - \tfrac{12}{6} m_1 = \tfrac{\Delta_1}{L} EI \\
&\text{GBC} \quad \tfrac{12}{3} m_3 + \tfrac{9}{3}(m_2+m_3) - \tfrac{9}{6}(m_2+m_3) - \tfrac{12}{6} m_4 = \tfrac{\Delta_2-\Delta_1}{L} EI \\
&\text{BCD} \quad \tfrac{12}{3} m_4 - \tfrac{12}{6} m_3 = \tfrac{\Delta_2-\Delta_1}{L} EI \\
&\text{Sc BG} \quad m_1 + m_2 - m_3 - m_4 = \tfrac{1}{2} \times 10{,}000 \cdot 12 \\
&\text{Sc CH} \quad m_3 + m_4 = \tfrac{1}{4} \times 20{,}000 \cdot 12
\end{aligned}
\right\} \quad . \quad . \quad (80a)
$$

EXAMPLE.—For the truss shown in *Fig.* 59, from equations (77) we get (80*a*), from which $m_1 = 62{,}000$ ft.-lb., $m_2 = 58{,}000$ ft.-lb., $m_3 = 20{,}220$ ft.-lb., and $m_4 = 39{,}780$ ft.-lb.

Vierendeel Trusses with Unsymmetrical or Inclined Chords.—If the upper and lower chords are not symmetrical about a longitudinal axis (when corresponding chord members are inclined or have different moments of inertia), the bending moments and shearing forces on the upper and lower members are

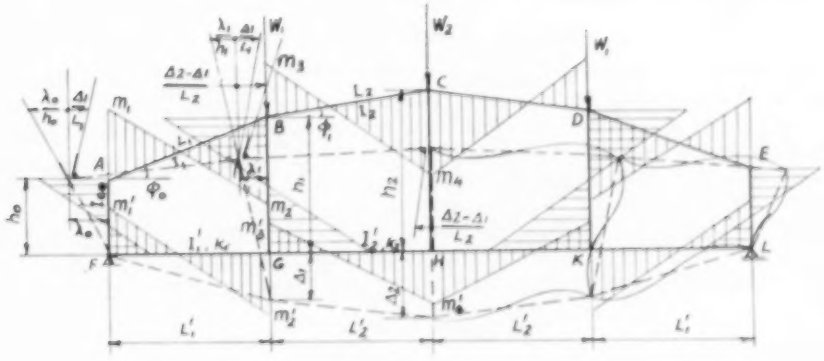

Fig. 60.

$$
\left.\begin{array}{ll}
\text{FAB} & m_1\alpha_{ab}+m_1\alpha_{af}-m_1'\beta_{af}-m_2\beta_{ab}=\dfrac{\Delta_1}{L_1}+\dfrac{\lambda_0}{h_0}\\
\text{ABG} & m_2\alpha_{ba}+(m_2+m_3)\alpha_{bg}-(m_2'+m_3')\alpha_{bg}-m_1\beta_{ba}=\dfrac{\Delta_1}{L_1}+\dfrac{\lambda_1}{h_1}\\
\text{GBC} & m_3\alpha_{bc}+(m_2+m_3)\alpha_{bg}-(m_2'+m_3')\beta_{bg}-m_4\beta_{bc}=\dfrac{\Delta_2-\Delta_1}{L_2}+\dfrac{\lambda_1}{h_1}\\
\text{BCD} & m_4\alpha_{cb}-m_3\beta_{cb}=\dfrac{\Delta_2-\Delta_1}{L_2}\\
\Sigma_c\,\text{BG} & \dfrac{m_1+m_2}{L_1}\cos\phi_0+\dfrac{m_1'+m_2'}{L_1'}-\dfrac{m_3+m_4}{L_2}\cos\phi_1-\dfrac{m_3'+m_4'}{L_2'}=W_1\\
\Sigma_c\,\text{CH} & \dfrac{m_3+m_4}{L_2}\cos\phi_1+\dfrac{m_3'+m_4'}{L_2'}=\dfrac{1}{2}W_2
\end{array}\right\}\quad\ldots\quad(81)
$$

not equal, and the points of contraflexure in the columns are no longer at mid-height. It is therefore necessary to have equations of equilibrium for joints at both the upper and lower chord. This may be avoided, and the number of equations considerably reduced, if it is assumed that the bending moments and shearing forces on each corresponding member are in the ratio of their respective stiffnesses. For the truss shown in *Fig.* 60 the bending moments on the lower chord can therefore be calculated from the relations $m'_1 = k_1 m_1$, $m'_2 = k_1 m_2$, $m'_3 = k_2 m_3$, $m'_4 = k_2 m_4$, in which $k_1 = \left(\dfrac{L_1}{L'_1}\right)^2 \cdot \dfrac{I'_1}{I_1}$ and $k_2 = \left(\dfrac{L_2}{L'_2}\right)^2 \cdot \dfrac{I'_2}{I_2}$.

For trusses with inclined members the horizontal translations λ of the joints (*Fig.* 60) due to vertical deflections Δ can also be taken into account when writing the equations of equilibrium. The values of λ do not represent separate unknowns, but can be expressed in terms of Δ as follows:

$$\lambda_1 = \frac{h_2 - h_1}{L'_2}(\Delta_2 - \Delta_1)\,; \quad \lambda_0 = \lambda_1 + \frac{h_1 - h_0}{L'_1}\Delta_1.$$

The equations of equilbirium can now be written in the usual way as in (81).

EXAMPLE.—Consider the four-bay truss with inclined top chord in *Fig.* 61. First calculate the coefficients k and the horizontal translations λ of joints A and B:

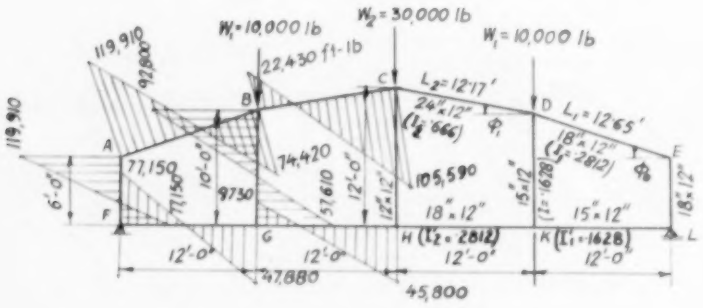

Fig. 61.

$$k_1 = \left(\frac{12{\cdot}65}{12{\cdot}0}\right)^2 \times \frac{0{\cdot}1628}{0{\cdot}2812} = 0{\cdot}6434\,; \quad k_2 = \left(\frac{12{\cdot}17}{12{\cdot}0}\right)^2 \times \frac{0{\cdot}2812}{0{\cdot}6666} = 0{\cdot}4338\,;$$

$$\lambda_1 = \frac{12 - 10}{12{\cdot}0}(\Delta_2 - \Delta_1) = 0{\cdot}1667(\Delta_2 - \Delta_1)\,;$$

$$\lambda_0 = 0{\cdot}1667(\Delta_2 - \Delta_1) + \frac{10 - 6}{12{\cdot}0}\Delta_1 = 0{\cdot}1667\Delta_1 + 0{\cdot}1667\Delta_2\,;$$

$$\cos\phi_0 = \frac{12{\cdot}0}{12{\cdot}65} = 0{\cdot}9486\,; \quad \cos\phi_1 = \frac{12{\cdot}0}{12{\cdot}17} = 0{\cdot}9860.$$

$$\left.\begin{aligned}
&\text{FAB}\ldots \frac{12{\cdot}65}{3{\cdot}0{\cdot}2812}m_1 + \frac{6{\cdot}0}{3{\cdot}0{\cdot}2812}m_1 - 0{\cdot}6434\frac{6{\cdot}0}{6{\cdot}0{\cdot}2812}m_1 - \frac{12{\cdot}65}{6{\cdot}0{\cdot}2812}m_2 = \frac{\Delta_1}{12{\cdot}65} + \frac{0{\cdot}1667\Delta_1}{6{\cdot}0} + \frac{0{\cdot}1667\Delta_2}{6{\cdot}0}\\
&\text{ABG}\ldots \frac{12{\cdot}65}{3{\cdot}0{\cdot}2812}m_2 + \frac{10{\cdot}0}{3{\cdot}0{\cdot}1628}(m_2 + m_3) - \frac{10{\cdot}0}{6{\cdot}0{\cdot}1628}(0{\cdot}6434m_1 + 0{\cdot}4338m_2) - \frac{12{\cdot}65}{6{\cdot}0{\cdot}2812}m_1 = \frac{\Delta_1}{12{\cdot}65} + \frac{0{\cdot}1667}{10}(\Delta_2 - \Delta_1)\\
&\text{GBC}\ldots \frac{12{\cdot}17}{3{\cdot}0{\cdot}666}m_3 + \frac{10}{3{\cdot}0{\cdot}1628}(m_2 + m_3) - \frac{10}{6{\cdot}0{\cdot}1628}(0{\cdot}6434m_1 + 0{\cdot}4338m_2) - \frac{12{\cdot}17}{6{\cdot}0{\cdot}666}m_4 =\\
&\qquad = \frac{\Delta_2 - \Delta_1}{12{\cdot}17} + \frac{0{\cdot}1666}{10{\cdot}0}(\Delta_2 - \Delta_1)\\
&\text{BCD}\ldots \frac{12{\cdot}17}{3{\cdot}0{\cdot}666}m_4 - \frac{12{\cdot}17}{6{\cdot}0{\cdot}666}m_3 = \frac{\Delta_2 - \Delta_1}{12{\cdot}17}\\
&\text{Sc.BG}\ldots \frac{m_1 + m_2}{12{\cdot}65} \times 0{\cdot}9486 + \frac{0{\cdot}6434}{12{\cdot}0}(m_1 + m_2) - \frac{m_3 + m_4}{12{\cdot}17} \times 0{\cdot}9860 - \frac{0{\cdot}4338}{12{\cdot}0}(m_3 + m_4) = 10{,}000\\
&\text{Sc.CH}\ldots \frac{m_3 + m_4}{12{\cdot}17} \times 0{\cdot}9860 + \frac{0{\cdot}4338}{12{\cdot}0}(m_3 + m_4) = \frac{1}{2} \times 30{,}000
\end{aligned}\right\} \quad (81a)$$

Substituting these values in equations (81) we obtain (81*a*), from which $m_1 = 119{,}910$ ft.-lb.; $m_2 = 74{,}420$ ft.-lb.; $m_3 = 22{,}430$ ft.-lb.; $m_4 = 105{,}590$ ft.-lb.;

$m'_1 = 77{,}150$ ft.-lb.; $m'_2 = 47{,}880$ ft.-lb.; $m'_3 = 9730$ ft.-lb.; and $m'_4 = 45{,}800$ ft.-lb.

Assuming that $E = 4{,}000{,}000$ lb. per square inch, then, $\Delta_1 = 0{\cdot}2572$ in., $\Delta_2 = 0{\cdot}4027$ in., $\lambda_1 = 0{\cdot}024$ in., and $\lambda_0 = 0{\cdot}1068$ in.

Continuous Vierendeel Trusses.

CONTINUOUS SYMMETRICAL TRUSSES.—Continuous symmetrical trusses, symmetrically loaded, may be analysed in a similar way to the trusses discussed previously. It is important that the deflected shape of the truss be clearly visualised and drawn to an exaggerated scale. This assists in setting out equations of equilibrium and in avoiding errors. The main difficulty in analysing continuous trusses lies usually in solving a large number of simultaneous equations. Although the deflections and sway entering into the equations may be easily eliminated, the number of simultaneous equations often remains large.

Examples of the analysis of continuous trusses are given to illustrate the procedure.

Two-span Symmetrical Trusses.—*Fig. 62 (b)* shows the deflected shape of a truss with angular gaps marked at each joint of the top chord. There is

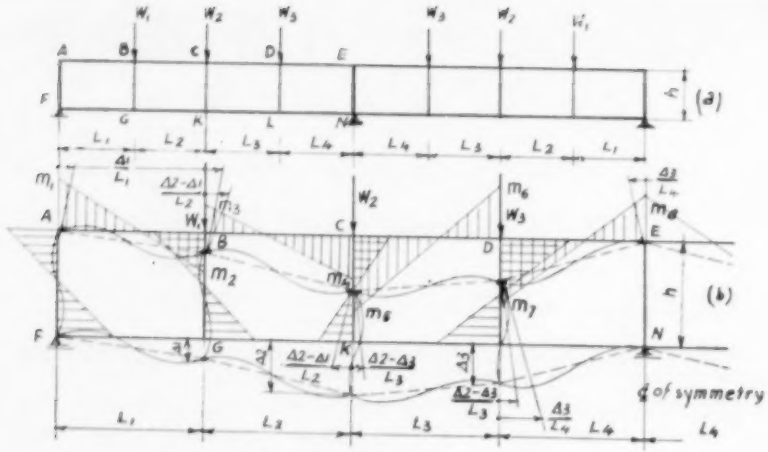

Fig. 62.

$$
\left.
\begin{aligned}
&FAB\ldots\ m_1\alpha_{ab}+m_1\alpha_{af}-m_1\beta_{af}-m_2\beta_{ab}=\frac{\Delta_1}{L_1}\\
&ABC\ldots\ m_3\alpha_{bc}-m_2\alpha_{ba}-m_4\beta_{bc}+m_1\beta_{ba}=\frac{\Delta_2-\Delta_1}{L_2}-\frac{\Delta_1}{L_1}\\
&ABG\ldots\ m_2\alpha_{ba}+(m_2+m_3)\alpha_{bg}-(m_2+m_3)\beta_{bg}-m_1\beta_{ba}=\frac{\Delta_1}{L_1}\\
&BCD\ldots\ m_4\alpha_{cb}+m_5\alpha_{cd}-m_6\beta_{cd}-m_3\beta_{cb}=\frac{\Delta_2-\Delta_1}{L_2}+\frac{\Delta_2-\Delta_3}{L_3}\\
&KCD\ldots\ m_5\alpha_{cd}+(m_3-m_4)\alpha_{ck}-(m_5-m_4)\beta_{ck}-m_6\beta_{cd}=\frac{\Delta_2-\Delta_3}{L_3}\\
&CDE\ldots\ m_7\alpha_{de}-m_6\alpha_{dc}-m_8\beta_{de}+m_5\beta_{dc}=\frac{\Delta_3}{L_4}-\frac{\Delta_2-\Delta_3}{L_3}\\
&CDL\ldots\ m_6\alpha_{dc}+(m_6+m_7)\alpha_{dl}-(m_6+m_7)\beta_{dl}-m_5\beta_{dc}=\frac{\Delta_2-\Delta_3}{L_3}\\
&DE\ \ldots\ m_8\alpha_{ed}-m_7\beta_{ed}=\frac{\Delta_3}{L_4}\\
&Sc\,BG\ldots\ \frac{m_1+m_2}{L_1}-\frac{m_3+m_4}{L_2}=\frac{1}{2}W_1\\
&Sc\,CK\ldots\ \frac{m_3+m_4}{L_2}+\frac{m_5+m_6}{L_3}=\frac{1}{2}W_2\\
&Sc\,DL\ldots\ \frac{m_7+m_8}{L_3}-\frac{m_5+m_6}{L_4}=\frac{1}{2}W_3
\end{aligned}
\right\}\quad\ldots\ (82)
$$

no horizontal sway, due to the geometrical symmetry of the truss and symmetry in loading. The equations of equilibrium are set out in the usual way as in (82).

EXAMPLE.—For the two-span truss shown in *Fig.* 63 and from equations (82) we have (82*a*),

from which $m_1 = 58{,}412{\cdot}9$ ft.-lb.; $m_2 = 55{,}015{\cdot}8$ ft.-lb.; $m_3 = 16{,}985$ ft.-lb.; $m_4 = 36{,}443{\cdot}7$ ft.-lb.; $m_5 = 42{,}284{\cdot}9$ ft.-lb.; $m_6 = 24{,}286{\cdot}4$ ft.-lb.; $m_7 = 53{,}552{\cdot}7$ ft.-lb.; and $m_8 = 73{,}018{\cdot}6$ ft.-lb.

Assuming that the cross-sectional dimension of all the members is 21 in. by

12 in., and E is 4,000,000 lb. per square inch, then $\Delta_1 = 0{\cdot}118$ in., $\Delta_2 = 0{\cdot}174$ in. and $\Delta_3 = 0{\cdot}102$ in. *Fig.* 63 shows bending moments, shearing forces, and axial forces.

$$
\begin{aligned}
&FAB \quad \tfrac{12}{3}m_1 + \tfrac{9}{3}m_1 - \tfrac{9}{6}m_1 - \tfrac{12}{6}m_2 = \tfrac{\Delta_1}{L}EI\\
&ABC \ldots \tfrac{12}{3}m_3 - \tfrac{12}{3}m_2 + \tfrac{12}{6}m_4 + \tfrac{12}{6}m_1 = \tfrac{\Delta_2-\Delta_1}{L}EI - \tfrac{\Delta_1}{L}EI\\
&ABG \ldots \tfrac{12}{3}m_2 + \tfrac{9}{3}(m_2+m_3) - \tfrac{9}{6}(m_2+m_3) - \tfrac{12}{6}m_1 = \tfrac{\Delta_1}{L}EI\\
&BCD \ldots \tfrac{12}{3}m_4 + \tfrac{12}{3}m_5 - \tfrac{12}{6}m_6 - \tfrac{12}{6}m_3 = \tfrac{\Delta_2-\Delta_1}{L}EI + \tfrac{\Delta_2-\Delta_3}{L}EI\\
&KCD \ldots \tfrac{12}{3}m_5 + \tfrac{9}{3}(m_5-m_4) - \tfrac{9}{6}(m_5-m_4) - \tfrac{12}{6}m_6 = \tfrac{\Delta_2-\Delta_3}{L}EI\\
&CDE \quad \tfrac{12}{3}m_7 - \tfrac{12}{3}m_6 - \tfrac{12}{6}m_8 + \tfrac{12}{6}m_5 = \tfrac{\Delta_3}{L}EI - \tfrac{\Delta_2-\Delta_3}{L}EI\\
&CDL \ldots \tfrac{12}{3}m_6 + \tfrac{9}{3}(m_6+m_7) - \tfrac{9}{6}(m_6+m_7) - \tfrac{12}{6}m_5 = \tfrac{\Delta_2-\Delta_3}{L}EI\\
&DE \quad \tfrac{12}{3}m_8 - \tfrac{12}{6}m_7 = \tfrac{\Delta_3}{L}EI\\
&Sc\,BG \ldots m_1 + m_2 - m_3 - m_4 = \tfrac{1}{2} \times 10{,}000 \times 12\\
&Sc\,CK \ldots m_3 + m_4 + m_5 + m_6 = \tfrac{1}{2} \times 20{,}000 \times 12\\
&Sc\,DL \ldots m_7 + m_8 - m_5 - m_6 = \tfrac{1}{2} \times 10{,}000 \times 12
\end{aligned}
\qquad \ldots (82a)
$$

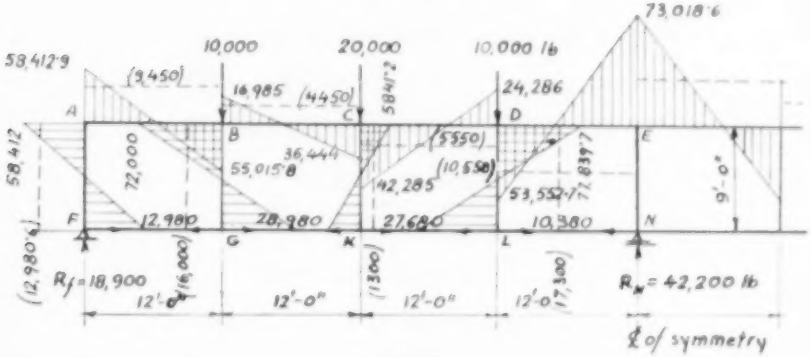

Fig. 63.

Three-span Symmetrical Truss.—The deflected shape of the truss and angular gaps are shown in *Fig.* 64 (*b*), and the equations of equilibrium are as in (83).

EXAMPLE.—For the three-span truss shown in *Fig.* 65 and from equations (83) we have (83*a*),

from which $m_1 = 19{,}927{\cdot}1$ ft.-lb.; $m_2 = 22{,}472{\cdot}7$ ft.-lb.; $m_3 = 12{,}728{\cdot}8$ ft.-lb.;
$m_4 = 4{,}921{\cdot}4$ ft.-lb.; $m_5 = 16{,}819{\cdot}5$ ft.-lb.; $m_6 = 30{,}830{\cdot}7$ ft.-lb.;
$m_7 = 65{,}138{\cdot}4$ ft.-lb.; $m_8 = 54{,}361{\cdot}6$ ft.-lb.; $m_9 = 20{,}567{\cdot}9$ ft.-lb.;
and $m_{10} = 39{,}432{\cdot}1$ ft.-lb.

Fig. 66 shows the distribution of bending moments, shearing forces, and axial

forces. The deflections, if required, can now be calculated from the equations of equilibrium. Assuming that the cross-sectional dimensions of all members are 21 in. by 12 in., and E is 4,000,000 lb. per square inch, then $\Delta_1 = 0{\cdot}036$ in., $\Delta_2 = 0{\cdot}021$ in., $\Delta_3 = 0{\cdot}113$ in., and $\Delta_4 = 0{\cdot}179$ in.

It is interesting to note, comparing the examples on pages 102 and 103 with

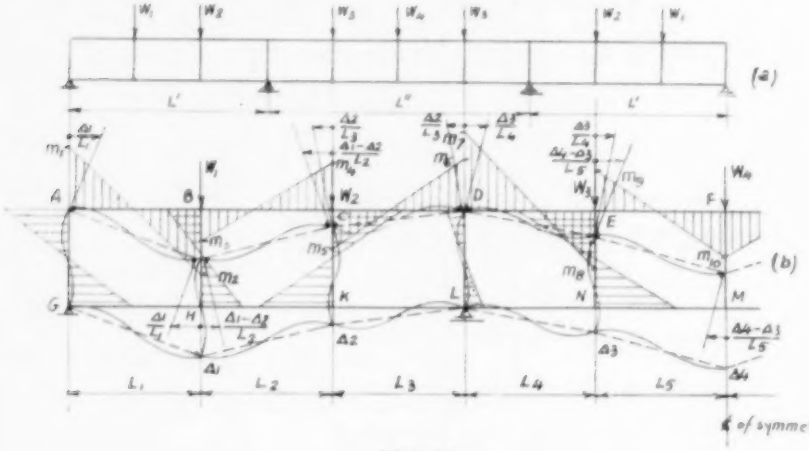

Fig. 64.

$$
\left.
\begin{array}{lll}
\text{GAB} & m_1\alpha_{aD} + m_1\alpha_{ag} - m_1\beta_{ag} - m_2\beta_{aD} = \frac{\Delta_1}{L_1} \\
\text{ABC} & m_2\alpha_{bB} + m_3\alpha_{bc} - m_1\beta_{bB} - m_4\beta_{bc} = \frac{\Delta_1}{L_1} + \frac{\Delta_1 - \Delta_2}{L_2} \\
\text{ABH} & m_2\alpha_{bB} + (m_2 - m_3)\alpha_{bh} - (m_2 - m_3)\beta_{bh} - m_1\beta_{bB} = \frac{\Delta_1}{L_1} \\
\text{BCD} & m_4\alpha_{cD} - m_5\alpha_{cd} + m_6\beta_{cd} - m_5\beta_{cD} = \frac{\Delta_1 - \Delta_2}{L_2} - \frac{\Delta_2}{L_3} \\
\text{BCK} & m_4\alpha_{cB} + (m_4 + m_5)\alpha_{ck} - (m_4 + m_5)\beta_{ck} - m_5\beta_{cB} = \frac{\Delta_1 - \Delta_2}{L} \\
\text{CDE} & m_6\alpha_{dc} + m_7\alpha_{de} - m_8\beta_{de} - m_5\beta_{dc} = \frac{\Delta_2}{L_3} + \frac{\Delta_3}{L_4} \\
\text{LDE} & m_7\alpha_{de} + (m_7 - m_6)\alpha_{dl} - (m_7 - m_6)\beta_{dl} - m_8\beta_{de} = \frac{\Delta_3}{L_4} \\
\text{DEF} & m_9\alpha_{ef} - m_8\alpha_{ed} + m_7\beta_{ed} - m_{10}\beta_{ef} = \frac{\Delta_4 - \Delta_3}{L_5} - \frac{\Delta_3}{L_4} \\
\text{NEF} & m_9\alpha_{ef} + (m_8 + m_9)\alpha_{en} - (m_8 + m_9)\beta_{en} - m_{10}\beta_{ef} = \frac{\Delta_4 - \Delta_3}{L} \\
\text{EF} & m_{10}\alpha_{fe} - m_9\beta_{ef} = \frac{\Delta_4 - \Delta_3}{L_5} \\
\text{Sc BH} & \frac{m_1 + m_2}{L_1} + \frac{m_3 + m_4}{L_2} = \frac{1}{2}W_1 \\
\text{Sc CK} & \frac{m_5 + m_6}{L_3} - \frac{m_3 + m_4}{L_2} = \frac{1}{2}W_2 \\
\text{Sc EN} & \frac{m_7 + m_8}{L_4} - \frac{m_9 + m_{10}}{L_5} = \frac{1}{2}W_3 \\
\text{Sc FM} & \frac{m_9 + m_{10}}{L_5} = \frac{1}{4}W_4
\end{array}
\right\} \quad . \quad . \quad (83)
$$

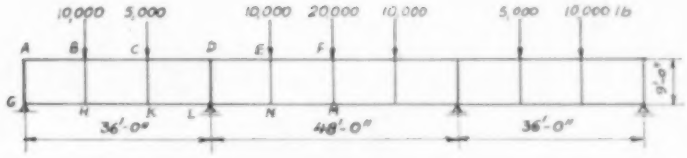

Fig. 65.

$$
\begin{aligned}
&GAB \ldots \tfrac{12}{3}m_1 + \tfrac{9}{3}m_1 - \tfrac{9}{6}m_1 - \tfrac{12}{6}m_2 = \tfrac{\Delta_1}{L}EI \\
&ABC \ldots \tfrac{12}{3}m_2 + \tfrac{12}{3}m_3 - \tfrac{12}{6}m_1 - \tfrac{12}{6}m_4 = \tfrac{\Delta_1}{L}EI + \tfrac{\Delta_1-\Delta_2}{L}EI \\
&ABH \ldots \tfrac{12}{3}m_2 + \tfrac{9}{3}(m_2-m_3) - \tfrac{9}{6}(m_2-m_3) - \tfrac{12}{6}m_1 = \tfrac{\Delta_1}{L}EI \\
&BCD \ldots \tfrac{12}{3}m_4 - \tfrac{12}{3}m_5 + \tfrac{12}{6}m_6 - \tfrac{12}{6}m_3 = \tfrac{\Delta_1-\Delta_2}{L}EI - \tfrac{\Delta_2}{L}EI \\
&BCK \ldots \tfrac{12}{3}m_4 + \tfrac{9}{3}(m_4+m_5) - \tfrac{9}{6}(m_4+m_5) - \tfrac{12}{6}m_3 = \tfrac{\Delta_1-\Delta_2}{L}EI \\
&CDE \ldots \tfrac{12}{3}m_6 + \tfrac{12}{3}m_7 - \tfrac{12}{6}m_8 - \tfrac{12}{6}m_5 = -\tfrac{\Delta_2}{L}EI + \tfrac{\Delta_3}{L}EI \\
&LDE \ldots \tfrac{12}{3}m_7 + \tfrac{9}{3}(m_7+m_8) - \tfrac{9}{6}(m_7+m_8) - \tfrac{12}{6}m_8 = \tfrac{\Delta_3}{L}EI \\
&DEF \ldots \tfrac{12}{3}m_9 - \tfrac{12}{3}m_8 + \tfrac{12}{6}m_7 - \tfrac{12}{6}m_{10} = \tfrac{\Delta_4-\Delta_3}{L}EI - \tfrac{\Delta_3}{L}EI \\
&NEF \ldots \tfrac{12}{3}m_9 + \tfrac{9}{3}(m_9+m_8) - \tfrac{9}{6}(m_9+m_8) - \tfrac{12}{6}m_{10} = \tfrac{\Delta_4-\Delta_3}{L}EI \\
&EF \ldots \tfrac{12}{3}m_{10} - \tfrac{12}{6}m_9 = \tfrac{\Delta_4-\Delta_3}{L}EI \\
&\Sigma c\,BH \ldots m_1 + m_2 + m_3 + m_4 = \tfrac{1}{2} \times 10{,}000 \times 12 \\
&\Sigma c\,CK \ldots m_5 + m_6 - m_3 - m_4 = \tfrac{1}{2} \times 5{,}000 \times 12 \\
&\Sigma c\,EN \ldots m_7 + m_8 - m_9 - m_{10} = \tfrac{1}{2} \times 10{,}000 \times 12 \\
&\Sigma c\,FM \ldots m_9 + m_{10} = \tfrac{1}{4} \times 20{,}000 \times 12
\end{aligned}
\qquad \ldots \quad (83a)
$$

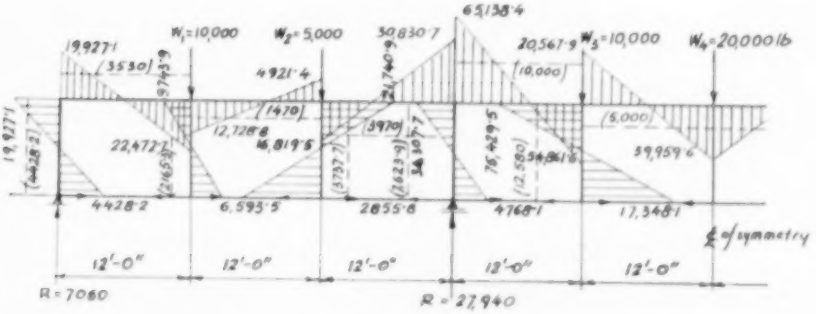

Fig. 66.

the example on page 99 (*Fig.* 59), that the effect of continuity on the bending moments and forces is not great. Although this must not be assumed to be general, it is in sharp contrast to the effect of continuity in continuous beams; the reason is that, in simply-supported Vierendeel trusses, continuity already exists between the chords and the columns. The continuity with other trusses modifies the bending moments and forces to a limited extent only. It should also be noted that the upper members, directly over the intermediate supports, may in certain cases be subject to an axial compressive force, in addition to the usual bending moments and shearing forces, while the corresponding lower members may still be in direct tension as in the case of single-span trusses. Continuity, therefore, will not always result in a change of sign of direct forces when compared with single-span trusses. The reason is that the bending moments and forces required to close the angular and linear gaps formed at the intermediate supports, should each span be treated as simply-supported, are not always of such magnitude as to exceed the initial stresses in the trusses.

(*Concluded.*)

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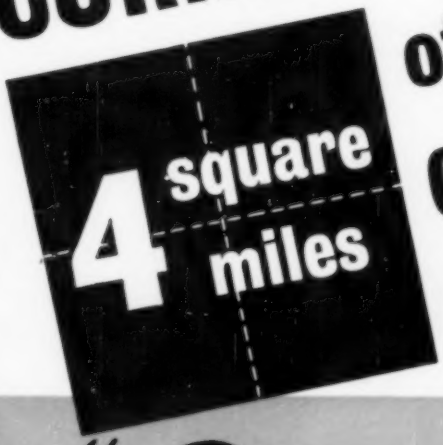

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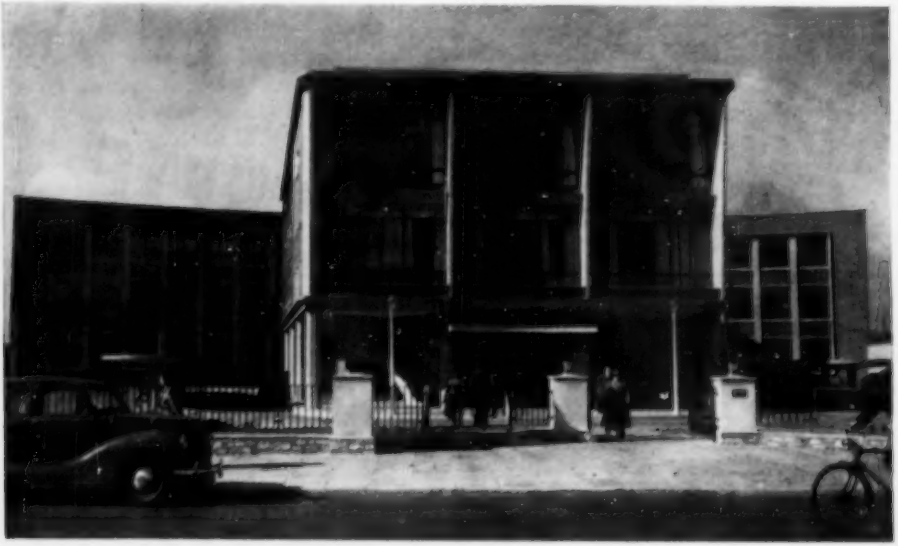

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A Prestressed Arch Bridge in Venezuela.

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THE bridge shown during construction in *Fig.* 1 is the longest of three being built in Venezuela to connect the port of La Guaira and Caracas. The design of the bridges is due to M. Freyssinet. The following notes are abstracted from "Engineering News-Record."

Corresponding deck beams and slabs, cross-beams, and spandrel columns for the three bridges are exactly the same in cross section. Variations in span are made by adding or removing panels of steel shuttering. Although the main spans vary as much as 45 ft., the cross sections of the arch ribs are identical. These ribs will be cast in the same shutters, modified as needed by the addition or omission of an element to make up for variations in span.

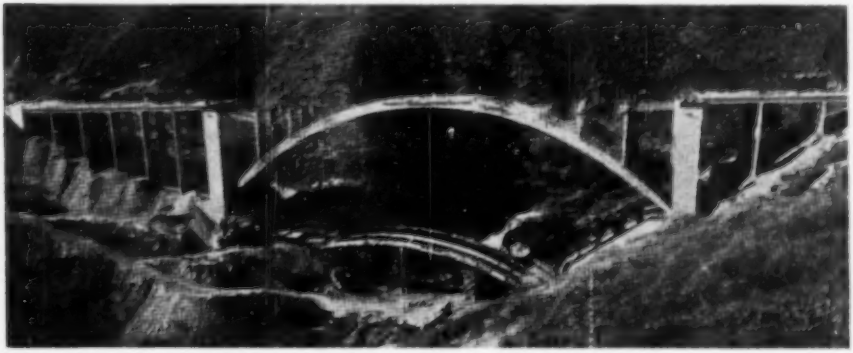

Fig. 1.

The longitudinal dimensions of the longest bridge are given in *Fig.* 2. The deck is 70 ft. wide and provides for six lines of traffic, three on each side of a central strip 4 ft. wide. The concrete surface is 2 in. thick at the edges and 7½ in. at the centre, the resulting slope of 1·5 per cent. being adequate for drainage. The footpaths are 2 ft. 4 in. wide, and are hollow to accommodate electrical and other services.

The bridge was designed in accordance with the American Association of State Highway Officials' H-20 loading. The high-tensile steel used for prestressing had an ultimate strength of 260,000 lb. per square inch and a working stress of 120,000 lb. per square inch. Since 0·2-in. diameter wires were employed in either 12-wire or 18-wire cables, the working tensile forces in the cables were 45,000 lb. and 68,000 lb. respectively. Concrete with strengths varying from 3300 lb. to 4000 lb. per square inch was required. A fairly high value of 50 lb. per square foot was used for the wind load because of the absence of reliable data.

The Deck.

The deck consists of precast slabs set between prestressed precast longitudinal beams spanning between cross-beams supported by the columns. A typical panel contains eight stringers at 9 ft. 2 in. centres, as shown in *Fig.* 3. These beams are 5 ft. 11 in. deep with webs 5½ in. thick except for a length of 4 ft. at each end, where they increase to 16 in., which is the width of the bottom flange. The beams are prestressed with three or four 12-wire cables (depending on the length of the beam) and are anchored at both ends with Freyssinet-type cones. Mild steel reinforcement is nominal, consisting of stirrups and a few longitudinal bars.

The precast slabs forming the deck are 5 ft. 3 in. long, 3 ft. wide, and 7 in. thick and are tied together transversely by cables spaced 3 ft. apart. These cables,

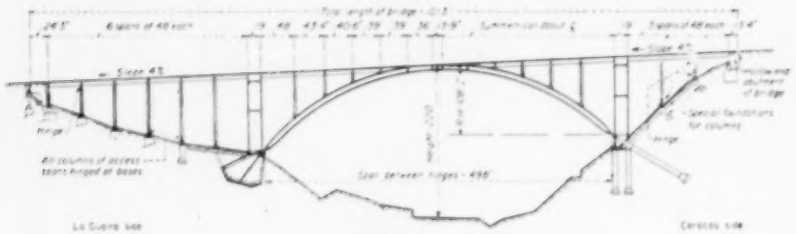

Fig. 2.

of eighteen 0·2-in. wires, pass through holes in the top flanges of the beam and rest in U-shape grooves formed in the slabs.

The deck is designed as three separate continuous systems, namely, one for the main arch and one for each of the approach viaducts. After all the beams, slabs, and cross-beams of one system were placed, prestressed transversely, and grouted, they were made to act as a continuous beam by longitudinal prestressing. Four or six 18-wire cables (four in the approaches, six over the arch) were placed in grooves in the tops of the beams.

The cross-beams, 9 ft. 2 in. deep, serve primarily to stiffen the deck transversely, but also support the two deck beams which are not carried directly by the spandrel columns. The top portions of the beams were cast in place between the ends of the stringers and are 8 in. wide and 5 ft. 11 in. deep. The bottom portions are inverted T-shaped sections and were precast in five separate elements. The webs are 8 in. thick and the bottom flanges 2 ft. 8 in. wide.

The precast elements were assembled and prestressed. After the deck was prestressed longitudinally, the cross-beams acted with the deck to resist the live load and wind load.

Spandrel Columns.

The spandrel columns are 10 ft. 6 in. wide and have 5½-in. webs. The widths of the flanges vary from 2 ft. 8 in. in the approach viaducts to 2 ft. over the arch. They were cast in place and then prestressed vertically. Those adjacent to the ends of the bridge and at the crown are hinged at the top since they are short, and if they were fixed they would be subjected to large bending moments due to longitudinal movement of the deck. The rest of the columns are rigidly connected to the cross-beams by prestressing cables. Columns in the approach viaducts are

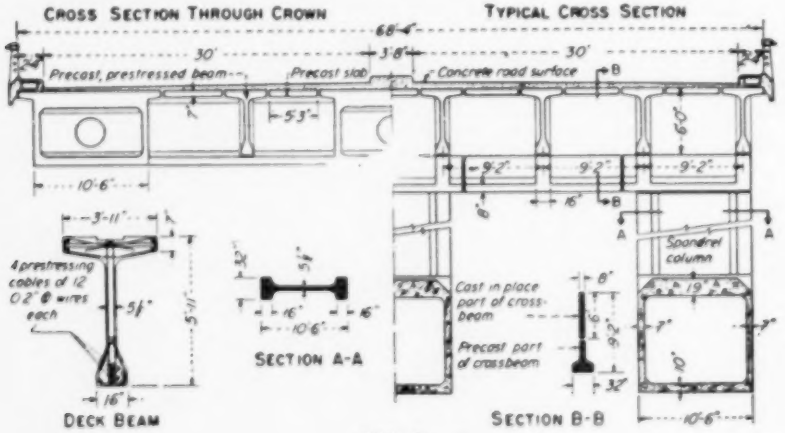

Fig. 3.

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hinged at the bottom and rest on precast hinge-blocks.

Piers.

Above each arch abutment is a pier, 137 ft. high, 80 ft. wide, and 20 ft. thick, comprising four 20-ft. by 20-ft. cells, each stiffened by horizontal concrete slabs at mid-height and at the top. The 4¾-in. thick vertical walls and partitions are designed as continuous slabs and contain only light reinforcement. The piers were cast in place and prestressed with ten 18-wire cables, one in each corner of the cells. Over each pier is a concrete rocker, which allows longitudinal movement of the deck.

Arch Ribs.

As the bending moment due to the live load was small compared with that due to the dead load, the shape of the arch was chosen so that the neutral axis would be as close as possible to the funicular curve of the dead load, except near the crown where the neutral axis was raised to increase the resistance of the arch transversely to wind forces. This caused tensile stresses at the crown which were counteracted by prestressing. The intrados of the arch is a smooth curve and the extrados is formed by straight lines connecting the bases of the columns, but to improve the appearance the extrados is chamfered. The arch has a theoretical rise of 108 ft. 2 in.

As shown in *Fig.* 3, the arch consists of three parallel ribs at 27-ft. 6-in. centres. Each is a hollow box 10 ft. 6 in. wide, except at the springing line where the width is increased to 17 ft. to provide increased transverse stability. There is no bracing between the ribs, except in the crown zone where ribs and deck join into one system.

The bottom slabs of the ribs are 10 in. thick and the top slabs 19 in.; the webs are 7 in. thick. The depth varies from 12 in. at the springings to a maximum of 10 ft. and then reduces to 6 ft. 6 in. at the crown. The reinforcement of the ribs is largely to resist shearing stresses in the ribs during construction and weighs about 85 lb. per cubic yard of concrete.

Arch Abutments.

The load of 6250 tons from the ribs is transmitted to the abutments through a

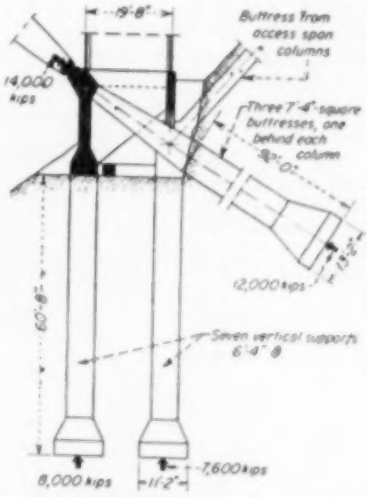

Fig. 4.—Open-type Abutment.

precast hinge-block weighing 13 tons and measuring 12 in. across its narrowest point. On the La Guaira side, the abutment is hollow and irregularly shaped. It is prestressed in three directions and only a small amount of mild steel reinforcement is used. On the Caracas side, decomposed schist made a large open excavation impossible. Instead, vertical wells and inclined tunnels were excavated to accommodate an open-type abutment (*Fig.* 4) consisting of seven vertical concrete supports 6 ft. 4 in. diameter and 62½ ft. deep, and three inclined buttresses 7 ft. 3 in. square and 100 ft. long, one behind each rib. The loads from the columns in the approach viaduct are also transmitted to this abutment through inclined concrete members near the ground so that excavation at the column bases could be reduced (*Fig.* 2).

Expansion and Contraction.

The maximum variation of temperature is not expected to exceed 30 deg. F. The deck is supported at the piers on concrete rockers that allow longitudinal movement but provide complete transverse stiffness. In the approach spans expansion and contraction are provided for by the flexibility of the columns, which are fixed at the tops and hinged at the bases; the columns nearest the ends of the bridge

are also hinged at the top to avoid bending moments. The deck is supported on concrete rockers at the end abutments and is fixed at the arch abutment pier.

Wind Forces.

Wind loads on the approach spans are resisted by transverse frames formed by the columns, cross-beams, and deck. Part of the wind load is transmitted to precast hinge-blocks at the column bases, which are considered to be fixed in the transverse direction, and part is resisted by the deck acting as a beam 70 ft. wide supported by the abutments and piers.

Part of the wind load on the arch is carried directly to the abutments through the ribs; the rest is transmitted to the deck at the junction of the ribs and the deck at the crown and through the columns, which are stiff transversely. The deck in turn acts as a beam 70 ft. deep and 504 ft. long, transmitting the wind forces through the rockers to the piers.

Construction.

The concrete mixture contained about 700 lb. of cement per cubic yard with a water-cement ratio between 0·4 and 0·43.

Precast hinge-blocks for the columns of the approach spans were placed from a 5-tons cableway spanning the site. The thin columns were cast alternately in 5-ft. lifts using steel shutters. During construction, horizontal timber bracing at 30-ft. intervals tied each column to the preceding one and finally to the end abutment. The same method was employed for the arch piers. For stabilisation during erection, the piers were tied to the abutments. Concrete was placed from 1½ cu. yd. buckets suspended from the cableway.

Three of the five pieces of the precast portions of the cross-beams were placed on the columns first; the last two were placed between and the joints were then grouted. They were prestressed with four 18-wire cables.

The deck beams and slabs for one span at a time, starting at the end abutments, were placed by a gantry crane that cantilevered 60 ft. in the direction of the bridge. About ten days were required to place the eight beams and 112 slabs for one span. Grouting of the joints and transverse prestressing followed immediately.

When all the approach spans were completed the top parts of the cross-beams were cast. Finally the decks in the three-panel and six-panel approach viaducts were made continuous by longitudinal prestressing, the cables being set in U-shaped grooves in the tops of the beams.

The precast beams and slabs were made in yards on each side of the gorge. The deck beams were precast in steel moulds strongly built in order not to deflect during the 100 uses expected of them for the three bridges. Eight 1½-kw. vibrators were placed along the bottom and fourteen 800-volt vibrators along each side—all having a frequency of 3000 cycles per minute.

The shutters were removed the day after placing the concrete, and on the fourth day the beams were prestressed. The heaviest beam weighed 24½ tons. Twenty-four slabs, weighing about 1100 lb. each, were cast each day. They were internally vibrated at 100 cycles per minute. As it was desired to place the arch rib hinges, weighing 12 tons each, from the 5-tons cableway, each hinge was precast in four pieces. When these were all in place, the 1½-in. joints between them were grouted and prestressed by four 18-wire cables.

Fig. 5 shows the procedure used for erecting the arches. The first portion of the centering for each rib, starting from the hinge, was assembled near one of the approach spans and placed with the 5-tons cableway. It comprised a platform about 30 ft. long carried on 3-in. by 10-in. timbers at 11 in. centres and weighing 5 tons. Each platform was suspended from two sets of cables (A and B) anchored in the abutment pier, each set being made up of 48 wires of 0·2-in. diameter. The ends of the cables were embedded in concrete cast around a transverse steel beam on which the platform rested.

The bottom of the arch rib was concreted first and acted with the timber platform in supporting the sides and top of the rib as it was concreted. Transverse bracing was placed between the ribs during construction.

The falsework platform for the second length of rib was exactly the same as the first, and was placed similarly and supported by two more sets of cables (C and D), one of 48 wires the other of 64 wires. Although the entire rib was cast only

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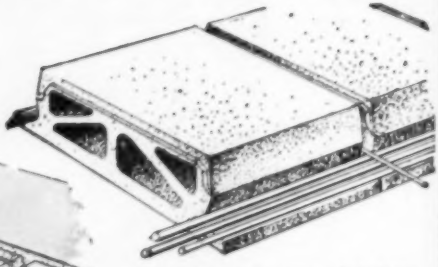

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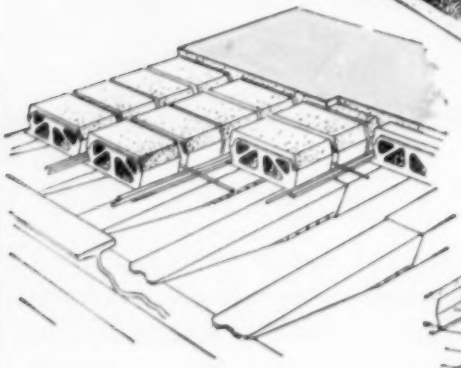

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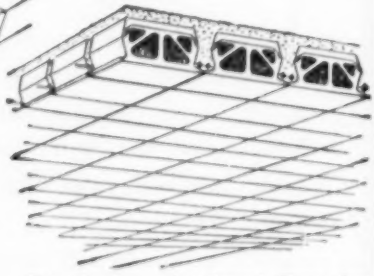

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the bottoms of the rib sections actually abutted, a small gap being left at the sides and top to form a temporary hinge so that the rib could be adjusted to its final curve.

The third centre for each rib was supported on a timber truss about 50 ft. long and weighing 10 tons. It was assembled in the valley below and hoisted into position with a hand winch set up on the first rib section and the cableway, and held by a set of 64-wire cables (E).

Only the bottom of the third rib section was cast before fixing the fourth section of centering. This was suspended from a set of 88-wire cables. After the bottom of the section was concreted, the side walls and thin strips of the tops of both the third and fourth rib sections were cast. hours. The ends of this section, encased in concrete as were all other butt joints, were supported by concrete blocks connected to the bottoms of the fourth rib sections. After the steel ties of the centering were released, the ribs for the first time began to act as arches.

Concreting then proceeded in a symmetrical manner. After the bottoms and sides of the ribs in the central span were cast up to the crown, hydraulic flat jacks and wedges were placed at the crown to regulate the level and the bending moments during the rest of the construction. There were fifteen 325-tons jacks and thirty-three 74-tons jacks bearing against the precast crown blocks. The shutters were stripped and lowered to the

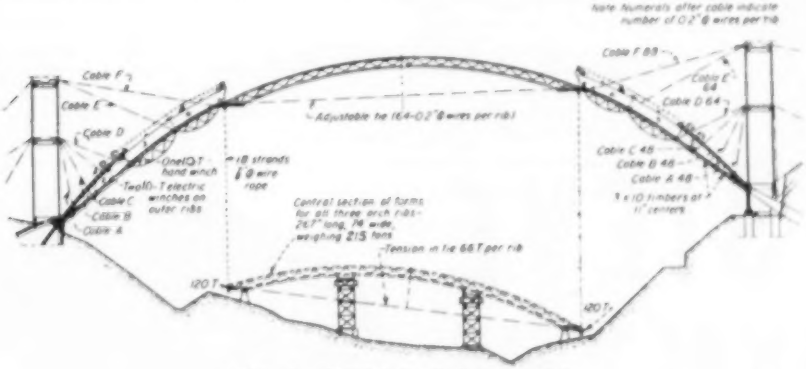
Note numerals after cable indicate number of 0.2" ϕ wires per rib
Cable F 88
Cable E 64
Cable D 64
Cable C 48
Cable B 48
Cable A 48
Cable F
Cable E
Cable D
Cable C
Cable B
Cable A
Adjustable to 16-0.2" ϕ wires per rib
One 10-T hand winch
Two 10-T electric winches on outer ribs
18 strands ϕ wire rope
Central section of forms for all three arch ribs - 26.7' long, 74' wide, weighing 215 tons
Tension in the 66 T per rib
120 T
120 T
9 x 10" timbers at 1' 11" centers

Fig. 5.—Erection of the Centres.

Temporary hinges, as already described, were left at the intersections of all the ribs.

The centering for the central section of the arch was made wide enough to accommodate all three ribs. It was assembled on the ground to form one shutter 265 ft. long weighing 215 tons and was erected from the rib sections already in place. Before raising the central section the gap between the first two rib elements at each end of the arch was grouted and the tops of these two rib sections tied together by prestressing cables in order that the ribs would act as beams during the hoisting of the central section. The first three hoisting cables were slackened and two 10-tons electric winches were set up on each of the outer ribs at the second-sections, with one hand winch on the central rib for compensation. The central section was lifted into place in eight ground, starting with the central member and proceeding towards the abutments. The spandrel columns were then cast, starting with those adjacent to the arch abutment column. The columns were braced longitudinally during casting, as were those in the approach spans. The cross-beams, deck beams, and slabs were placed by a gantry crane, as were their counterparts in the approaches.

After completion of all the concreting there were 3-in. gaps in each of the ribs at the crown. These were grouted and the central sections of the ribs prestressed. Finally the gaps between the rib sections were grouted and connected as were those between rib lengths Nos. 1 and 2.

The bridges are being erected for the Venezuelan Ministry of Public Works by Empresas Campenon-Bernard de Venezuela.

Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Concrete Placing and Formwork. By A. B. Harman. Technical College, Church Street, Barnsley. 16 February. 7.15 p.m.

Arbitration in Building Disputes. By N. P. Greig. Medway College of Technology, High Street, Chatham. 14 February. 7 p.m.

Alternative Forms of Construction. By L. R. Creasy. Technical College, Denzil Road, London, N.W.10. 14 February. 7 p.m.

Repair of Dangerous Buildings. By F. L. Felgate. Technical College, Tonbridge Road, Maidstone. 15 February. 7.15 p.m.

Thermal Insulation of Buildings. By R. R. Houston. Technical College, Newport, Isle of Wight. 15 February. 7 p.m.

Prestressed Concrete. By F. Walley. Carnegie Hall, Abington Street, Northampton. 16 February. 7.15 p.m.

Essentials of Good Concreting. By E. E. H. Bate. Gas Showrooms, Sandgate Road, Folkestone. 16 February. 7 p.m.

Structural Problems of Multiple-Story Housing. By W. A. Fairhurst. Courtroom, City Chambers, Edinburgh. 17 February. 7.15 p.m.

That the Greater Use of Alternative Materials would give Increased Productivity in the Building Industry. Discussion. Kirby Grammar School, Middlesbrough. 17 February. 7 p.m.

An Introduction to Prestressed Concrete. By R. C. Blyth. Gas Showrooms, Osborne Street, Grimsby. 23 February. 7.15 p.m.

Advantages of Thermal Insulation of Buildings. By J. S. Alton. Marine and Technical College, South Shields. 24 February. 7 p.m. By A. M. Palmer. 95 Tenison Road, Cambridge. 28 February. 7.30 p.m.

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SITUATION VACANT. The British Reinforced Concrete Engineering Co., Ltd., have vacancies for reinforced concrete designers and detailers, with experience, in their Stafford, London, Liverpool, Bristol, Newcastle-on-Tyne, and Glasgow offices. Staff pension scheme and five-days' week. Apply in writing to CHIEF ENGINEER, Stafford.

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SITUATIONS VACANT. Draughtsmen/Designers required to H.N.C. standard for our special steel formwork division. Experience in designing steel formwork for concrete structures an advantage, but not essential. Experience in work of similar nature is adequate providing applicants have initiative and the will to adapt their knowledge to a field continually growing in importance. Applicants must be prepared to undertake occasional tours overseas. Exceptional scope for advancement in a company with a reputation for rewarding merit. Apply, giving full details, to : Secretary to MANAGING DIRECTOR, ACROW (ENGINEERS) LTD., South Wharf, Paddington, London, W.2.

REINFORCED CONCRETE ENGINEERS, DESIGNERS, & DRAUGHTSMEN, CIVIL ENGINEERS, STRUCTURAL STEEL DESIGNERS

required in Manchester and Nottingham for interesting variety of prestressed and reinforced concrete buildings and industrial structures. Permanent positions, staff pension scheme. Apply, with full details of experience and education, to :

MATTHEWS & MUMBY, LTD., 129, Stockport Road, Manchester 12.

SITUATIONS VACANT. Senior engineer, with wide experience of reinforced concrete design, required for Bristol office for work on interesting projects. Also designer-detailers. Permanent positions with good prospects. Apply, giving details of age, qualifications, and experience. CLARKE, NICHOLLS & MARCELL, Berkeley Cottage, Berkeley Square, Bristol.

SITUATION VACANT. North Thames Gas Board. A senior draughtsman, for the design and detailing of reinforced concrete structures and general engineering work is required in the chief engineer's department, Westminster. Candidates should preferably be Corporate Members of the Institutions of Civil or Structural Engineers. Starting salary within the range of £745 to £895 per annum, according to age, qualifications, and experience. The successful candidate will be required to join the staff pension scheme. Applications, giving age and full particulars, should be sent to the STAFF CONTROLLER, NORTH THAMES GAS BOARD, 30 Kensington Church Street, London, W.8, quoting reference 666/197.

SITUATIONS VACANT. Air Ministry require in London structural engineering designer/draughtsmen in Works Department for reinforced concrete or structural steelwork with sound technical training and several years' varied experience in design and detailing of (a) Reinforced concrete construction for all types of buildings, or (b) steel-framed sheds, warehouses, and similar buildings. Salaries up to £810 per annum, starting pay dependent upon age, qualifications and experience. Extra-duty allowance or overtime payable. Promotion prospects. Posts non-pensionable with long term possibilities. Natural born British subjects only. Write, stating age, qualifications, employment details, including type of work done, to MINISTRY OF LABOUR, 236 Walworth Road, London, S.E.17, quoting Order 82.

SITUATIONS VACANT. Tarmac Limited require the following staff at head office. (1) Senior estimator for concrete products, able to read drawings and prepare own quantities. Up-to-date experience essential. (2) Architectural draughtsmen, fully qualified and able to work with the minimum of supervision. These appointments are permanent and well paid. Write, giving full details of experience and salary required, to CLERICAL MANAGER, TARMAC, LTD., Ettingshall, Wolverhampton.

SITUATION VACANT. Senior designer-draughtsman required for London headquarters of well-known reinforced concrete engineers specialising in hollow-tile type floor construction. Permanent, progressive, and pensionable post. Alternate Saturdays. Write fully qualifications, experience, and salary required, to BOX 4114, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Consulting engineers in N.W. London require an absolutely first-class reinforced concrete detailer capable of working entirely without supervision. A commencing salary up to £800 per annum will be paid to the right man. Every encouragement will be given for his future progress. Five-days' week, pension scheme, and luncheon club. BOX 4118, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

(Continued on page lxi.)

Symposium on the Observation of Structures.

A SYMPOSIUM on the observation of structures, sponsored by the Réunion Internationale des Laboratoires d'Essais et de Recherches sur les Matériaux et les Constructions, will be held in Lisbon in October 1955. The subjects for discussion are the use of measuring instruments, determination of the properties of materials, loads acting on structures, and similar problems. Full details may be had from the Secretary of the Symposium, Laboratório Nacional de Engenharia Civil, Av. do Brazil, Lisboa, Portugal.

MISCELLANEOUS ADVERTISEMENTS.

(*Continued from page lx.*)

SITUATION VACANT. Civil engineering assistant required by Westminster consulting engineer. Graduate with 2–3 years' experience in reinforced concrete, steelwork, etc. Capable of assuming authority. Salary £600–£700. Particulars to BOX 4116, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Civil Engineering draughtsman required by consulting engineers in Westminster. Educational qualifications: Regular course of studies for Higher National Certificate or equivalent. Previous experience of reinforced concrete detailing desirable. Salary will be according to experience. BOX 4122, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Reinforced concrete designer-detailer required by Westminster firm of consulting engineers. Post is progressive both as to position and remuneration. Salary £600 upwards based on experience and technical qualifications. Details, please, of experience, employment, etc. BOX 4123, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Engineer or research assistant wanted with concrete background for research department of large building concern to help organise and run quality control of concreting on building sites and in factories. Headquarters at Feltham. Five-days' week. Good pension scheme. Apply, with summary of experience and qualifications and stating salary required, to BOX 4119, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Well-known firm of reinforced concrete engineers and contractors require designers and detailers for their London office. Previous experience in frames and floors an asset. Permanent and progressive posts. Salary by arrangement. Write, giving full details of age, qualifications, and experience. BOX 4120, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Technical assistant to director. Age 21–28, graduate or qualified with some drawing office and design experience. Good prospects for young man with initiative and experience in technical estimating, etc., for all types of reinforced concrete structures. London, W.1, district. Full details to BOX 4124, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Brian Colquhoun & Partners require a thoroughly experienced and competent civil and structural design engineer in their London office. Age 35–40 years with at least 15 years' experience in design office. Starting salary £950 per annum. Also several reinforced concrete designer-detailers with at least 10 years' experience in design office. Salary range £750 to £850 per annum. Applications in writing only, stating full details of experience and technical qualifications, to BRIAN COLQUHOUN & PARTNERS, 18 Upper Grosvenor Street, London, W.1.

THE HELICAL BAR & ENGINEERING CO., LTD., 82 Victoria Street, London, S.W.1, require the following staff for their London Office:

SENIOR DESIGNER DRAUGHTSMAN. For reinforced concrete light-framed structures and hollow tile construction.

DRAUGHTSMAN/DETAILER.

QUANTITY SURVEYOR. To take-off and prepare bills of quantities for reinforced concrete work.

JUNIOR TAKER-OFF.

Pension scheme. Alternate Saturdays free. Apply in writing, giving brief details of experience and salary required.

SITUATIONS VACANT. Draughtsmen-detailers (all grades) required. Good experience in reinforced concrete essential. Five-days' week. Modern office conditions. Also technical assistant for dealing with projects, correspondence, etc. Full details and starting salary to JOHN LIVERSEDGE & ASSOCIATES, Consulting Engineers, 42 Portland Place, London, W.1.

SITUATION VACANT. Reinforced concrete designer/architectural draughtsman required for interesting and varied work on all types of reinforced, precast, prestressed, and composite structures, etc. Preferably with some knowledge of the fundamentals of prestressed concrete design. Apply in confidence, stating experience and salary required, to GEO. MOSS & SONS, LTD., Pennington Saw Mills, Leigh, Lancs.

SITUATION VACANT. The Bowater Paper Corporation Limited have a vacancy for a Civil Engineering Design Draughtsman. Applicants should have either passed Sections A and B Examination of the Institution of Civil Engineers or have obtained exemption. Practical experience should include reinforced concrete and structural steelwork design, layout of railways, roads, drains, and sewers. Knowledge of surveying and setting-out an advantage but not essential. Contributory pension scheme. Applications will be treated confidentially. They should state details of age, education, training, experience, and salary required and be sent to the CHIEF ENGINEER, THE BOWATER PAPER CORPORATION LTD., Northfleet, Gravesend, Kent.

FOR HIRE.

FOR HIRE. Lattice steel erection masts (light and heavy), 30 ft. to 150 ft. high, for immediate hire. BELLMAN'S, Terminal House, London, S.W.1. Telephone: Sloane 5259.

(*Continued on page 114.*)

DATA FOR PRICING REINFORCED CONCRETE.

Materials.

(Delivered in London area.)

AGGREGATES (per cu. yd.).—Washed sand, 23*s.* 8*d.* Clean shingle: ¾ in., 20*s.*; ½ in., 22*s.* 7*d.* Pit ballast, 23*s.* 1*d.*

CEMENT (per ton, delivered at Charing Cross).—Portland cement, 6 tons and upwards, 96*s.* 6*d.* 1 ton to 6 tons, 105*s.* 6*d.* Paper bags and non-returnable jute sacks included.
Rapid-hardening Portland, 8*s.* above ordinary Portland.
Aquacrete and 417, 32*s.* 6*d.* above ordinary Portland; paper bags included.
Colorcrete (buff, red, and khaki), in 6-ton loads, 138*s.*; paper bags included.
Snowcrete, £12 17*s.*, inc. paper bags.
"Super-Cement," 32*s.* 6*d.* per ton above ordinary Portland cement; paper bags included.
High-alumina cement, 1 ton and upwards, 280*s.* 9*d.* per ton.
Snowcem paint, 71*s.* per cwt. including containers.

SHUTTERING.—1" and 1½" deal sawn shuttering boards, £95–£100 per standard.

REINFORCEMENT.—Mild steel bars, B.S. 785 (per cwt.); ¾ in. to 2¾ in., 43*s.* 3*d.*; ⅝ in. to ½ in., 45*s.*; ⅜ in., 45*s.* 9*d.*; ¼ in., 47*s.* 3*d.*

Materials and Labour.

(Contracts up to £5000. Including 10 per cent. profit.)

PORTLAND CEMENT CONCRETE, 1 : 2 : 4.—
Foundations, 2*s.* 5½*d.* per cu. ft. Columns, 3*s.* 4½*d.* per cu. ft. Beams, 3*s.* 1½*d.* per cu. ft. Floor slabs, 4 in. thick, 8*s.* 10½*d.* per sq. yd.; Do., 5 in., 11*s.* 2*d.*; Do., 6 in., 12*s.* 9½*d.*; Do., 7 in., 15*s.* Walls 6 in. thick, 14*s.* 3*d.* per sq. yd. Add for hoisting above ground level, 3*s.* 8*d.* per cu. yd. Add for rapid-hardening Portland cement, 2*s.* 6*d.* per cu. yd.

REINFORCEMENT.—Mild steel bars (B.S. 785), including cutting, bending, fixing, and wire (per cwt.)—¾ in. to ⅞ in., 74*s.* 8*d.* ⅝ in. to ½ in., 68*s.* 7*d.* ⅜ in. to 2¼ in., 62*s.* 10*d.*

SHUTTERING AND SUPPORTS.—
Walls, 207*s.* 6*d.* per square.
Floors (average 10 ft. high), 192*s.* 3*d.* per square. In small quantities, 2*s.* 6½*d.* per sq. ft.
Columns, average 18 in. by 18 in. (per sq. ft.), 2*s.* 11*d.*; in narrow widths, 3*s.* 8½*d.*
Beam sides and soffits, average 9 in. by 12 in. (per sq. ft.), 2*s.* 10½*d.*; in narrow widths, 3*s.* 4½*d.*
Raking, cutting, and waste, 6*d.* per lin. ft.
Labour on splays, 3*d.* per lin. ft.
Small fillets to form chamfers, 6*d.* per lin. ft.

Wages.

The rate of wages on which the above prices are based are: Carpenters and joiners, 3*s.* 10½*d.* per hour (carpenters 2*d.* a day tool money); Labourers, 3*s.* 4*d.*; Men on mixers and hoists, 3*s.* 7*d.*; Bar-benders, 3*s.* 7½*d.*

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MISCELLANEOUS ADVERTISEMENTS.

(Continued from page 113.)

WEXFORD COUNTY COUNCIL
REPUBLIC OF IRELAND
CONSTRUCTION OF REINFORCED CONCRETE BRIDGE

The time for submission of tenders for the construction of a reinforced concrete bridge over the River Slaney at Wexford has been extended to *12 o'clock noon on Monday, 14th March, 1955.*

THOMAS F. McDERMOTT,
Secretary,
Wexford County Council.

County Hall,
Wexford.
14th January, 1955.

PLANT FOR SALE.

"REICH" VIBRATION BLOCK-MAKING MACHINE Type H3A.
No. 5763. Electric motor 400/690 volts, 3 phase, 50 cycles. No. 8358.
100 metre long LINE CONVEYOR, 400/440 volts. Motor No. 102264.
Reich Counter-current MIXER, Type 375.GZ., No. 5755, with built-in 13 h.p. electric motor 400-v., 3-phase, No. 8424.
Model E.3 Eonit modified CEMENT PLANT with swivel chute, 10/14 tons capacity. 500 lb. weigh dial. 400-v., 3-phase. Motor No. 7950201.
STACKER complete with electric KING HOIST No. PB8081.
STACKER complete with electric KING HOIST No. PB8080.
STACKER complete with electric KING HOIST No. PB8197.
STACKER complete with electric KING HOIST No. PB8198.
Two Vibromax VIBRATORS Type A400/1000, 400-v., A.C., complete with gland packings, table buffers, contact switches, and release mechanism.
Three sets of VIBRATOR WEIGHTS.
King ¼-ton Electric HOIST No. PB8849.
" " " " No. PB9668.
¼ mile of 40-lb. F.B. RAIL.

AVAILABLE FOR IMMEDIATE INSPECTION

BOX 4121, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

DESIGN WORK WANTED. Reinforced concrete designer-draughtsman with 20 years' experience in the design of reinforced concrete structures, willing to assist consulting engineers in foundations for heavy industrial structures, chemical plant, etc., undertakes to prepare complete designs, calculations and working drawings. Excellent references. Moderate fee. BOX 4117, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

PATENT.

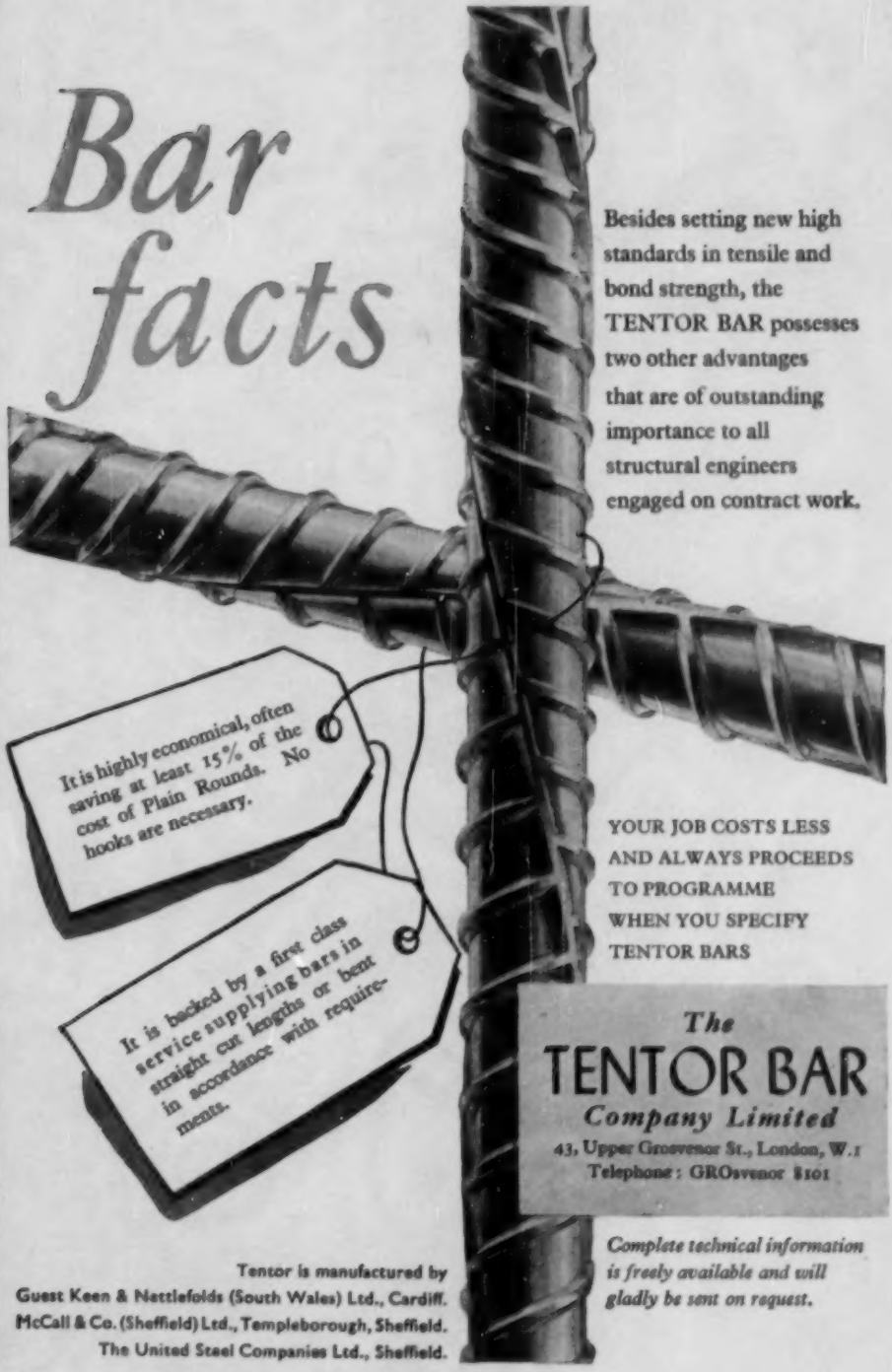
PATENT. The Proprietors of British Patent No. 643,685 for "IMPROVEMENTS IN REINFORCED BRICK BEAMS" desire to enter into negotiations with a firm or firms for the sale of the patent, or for the grant of licences thereunder. Further particulars may be obtained from MARKS & CLERK, 57 & 58 Lincoln's Inn Fields, London, W.C.2.

FOR SALE.

FOR SALE. Steel plates to sizes, pressed sections, discs, and general metalwork. Keen prices and delivery. E. STEPHENS & SON, LTD., Bath Street, London, E.C.1.

FOR SALE. Steel buildings, small and large, home or abroad, 30 ft. up to 200 ft. wide. Large clear high spans with or without runways, etc., for lifting loads. BELLMAN HANGARS, LTD., Terminal House, London, S.W.1.

Bar facts



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It is highly economical, often saving at least 15% of the cost of Plain Rounds. No hooks are necessary.

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Tel : 24455

GLASGOW
25 Newton Place
Charing Cross, C.3
Tel : Douglas 5906

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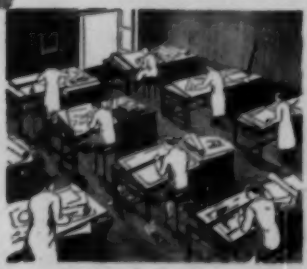

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